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Effect of live load on soil-steel structure under shallow cover.

Shantaram G. Ekhande
University of Windsor

EFFECT OF LIVE LOAD ON SOIL-STEEL STRUCTURE UNDER SHALLOW COVER

BY

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To My Parents

ABSTRACT

In this investigation two conduit models of different degrees of flexibility are tested to study the three dimensional effects of a concentrated (e.g. wheel) load under the conditions of shallow cover.

From the test data the live load dispersion in the soil is determined and compared with the load dispersion specified by the OHBDC and AASHTO.

An expression is also proposed for calculating the thrust in the conduit wall and the thrusts calculated by the proposed equation are compared with the experimental values and the estimated values by OHBDC and AASHTO code provisions.

The bending moment variation in the conduit is also studied and the minimum height of backfill to eliminate the bending moments in a conduit due to the concentrated load is suggested.

In addition to above, the deflection formula has been checked for the conduit under shallow backfill.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials.
Ap	Area under the pressure envelope. lb./in.
b	Actual width of loading plate. inch.
2bc	Equivalent load dispersion width in the longitudinal direction, at the crown level. inch.
be	Equivalent width of loading plate to give uniform pressure below it. inch.
c	Distance of extreme fiber from the neutral axis, in.
D	Diameter(span) of the conduit. inch.
Dl	Deflection lag factor to compensate for the volume change of soil with time, dimensionless
DEL($\triangle$), $\triangle x$	Deflection of conduit by Iowa equation, inch.
$\triangle_1$ ($\triangle_2$)	Deflection by first (second) modification, inch.
E	Modulus of elasticity of conduit material, psi.
E'	Modulus of soil reaction, psi.
E'm	Modified modulus of soil reaction, psi.
H, h	Depth of soil cover, inch.
H*	Net depth of soil cover, inch.
I	Moment of inertia of conduit wall, $in.^4/in.$
K	Bedding constant which varies with the angle of bedding, dimensionless.
L	Length of conduit, inch.
M, Ma, Mb, Mx	Bending moment in conduit wall, lb.in.

N_θ	Axial thrust at θ degree location, lb/in.
OHBDC	Ontario Highway Bridge Design Code.
P, P_actual	Concentrated load, lb.
P'	Load per unit length of conduit= P/2bc, lb./in.
p, pmax, pz	Soil pressure at the crown level, psi.
qc	Measured soil pressure at the crown level, psi.
R, r	Radius of conduit, inch.
T	Axial thrust, lb/in.
T-1	Thrust by OHBDC = p.r, lb./in.
T-2	Thrust by OHBDC ≐ (P/2bc)x1/2=P'/2, lb./in.
t	Thickness of conduit wall, in.
W	Load on pipe per unit length, lb./in.
w	Actual length of loading plate, inch.
2wc	Equivalent load dispersion width in the span direction of the conduit at crown, in.
we	Equivalent length of loading plate to give uniform pressure below it, in.
θ	Angle made with the vertical through the centre of the conduit, degrees.
π_1, π_2, π_3	Nondimensional numbers.
m	Poisson's ratio, dimensionless.
E,Ein, Eout	Strain in conduit fibers, micro.in/in.
Eav	Average strain or strain due to axial thrust, micro in/in.
Ebend	Strain due to bending, micro in/in.
Alpha(α)	Load dispersion angle in the longitudinal direction, degrees.

α_{av}	Average of the load dispersion angles alpha for one soil cover, degrees.
Beta (β)	Load dispersion angle in the span direction of conduit, degrees.
β_{av}	Average of the load dispersion angles beta for one soil cover, degrees.

CHAPTER I

INTRODUCTION

1.1 General

Underground flexible conduits are being constructed frequently as highway and railway bridges and have proven to be practical and more economical structures than the conventional bridges made of Reinforced Concrete, Prestressed Concrete or Structural Steel.

A soil steel structure is a corrugated steel pipe embedded in soil. The high load carrying capacity of such structures is basicaly due to the soil-structure interaction, which activates semi-passive earth pressure at the sides of the flexible culvert. Many years of field experience have shown that the culverts under high backfills carry loads mainly through ring compression(31,32), although bending moments may develop that cause yielding or formation of plastic hinges in the culvert wall. The deformations remain small provided the culvert is covered by a sufficient depth of good quality granular backfill, and that the culvert is capable of carrying the imposed ring compression forces. A Typical soil-steel structure is shown in Fig. 1.1

There are several advantages of soil-steel structures over the conventional bridge structures. These are briefly as follows:

(i) Normal embankment construction eliminates construction or narrowing of roadway section and hence greater safety.

(ii) No bridge deterioration problem due to salt corrosion and eliminates winter icing and other problems associated with temperature fluctuations. All in all, little or no maintenance compared to the conventional bridges.

(iii) On site erection and compaction: Easy and economical to construct.

(iv) Less design and construction time: Closed and/or detour are of short duration.

(v) Has proved economical and feasible alternative where bearing capacity is poor for bridge piers and/or abutments. Also it eliminates problem of the fill settlement beside the abutments.

(vi) It has favourable aesthetics. Natural appearance of earth slopes and plant cover preserves the environment.

(vii) It is easily widened. Increased lane capacity is obtained by simple extension of ends and wider fills.

The 'soil-steel structures' are generally large diameter circular pipes, vertical or horizontal ellipses, arches or pipe arches. Some of these common shapes are shown in Fig.1.2. Most culvert structures have spans in the range from few inches to 45 ft. and the largest has a span of 52 ft.(31)

The culvert structures are constructed of a corrugated metal (aluminium or steel) structural plates from 0.1 inch to 0.3 inch thick. Stiffener beams or angles are frequently used to increase the flexural stiffness and moment capacity of the structural plate of the upper zone in the long span. These are curved to confirm to the shape of the culvert and are bolted to the structural plate at intervals along the length of the structure.

1.2 Historical Review

The use of underground flexible conduits dates back to the turn of the century. In 1896 James Watson and Stanley Simpson jointly perfected and patented a design for a culvert made from corrugated steel sheets. Engineers began to understand that the flexible steel culvert pipe was not acting by itself, but in co-operation with the surrounding soil envelope. The concept of composite action of materials is widely used in contemporary construction. We have composite materials such as reinforced concrete or reinforced plastic and we have composite products such as stressed skin plywood panels or concrete filled steel pipe piles. Engineers are finding new ways to combine the best properties of various materials to achieve the most efficient design.

In the early years of development, the flexible soil-steel structure was relatively small, functioning adequately as a sewer or culvert in various highway, railway and municipal applications. Structural design was based on empirical equations derived by measuring and correlating deflection, height of fill, pipe diameter and steel thickness. Assuming that the failure occured when the deflection reached 20 % of diameter, a design value of

5 % of diameter was used giving a safety factor of 4. This safety factor was also sufficient to compensate for variations in the type of soil used for backfill and the method in which it was compacted. No attempt was made to correlate soil characteristics with structural design, although specific installation techniques were recommended by the pipe manufacturers.

In the early 1930's field bolted structural plates with larger corrugations and heavier gauges were introduced as a flexible soil steel pipe structure. Starting at 5 ft. in diameter, these structures complemented the standard corrugated steel pipe and enabled the construction of structures upto 15 ft. in diameter. The same empirical method of design was used but with an increased emphasis on the selection and proper compaction of soil used for backfill.

The method of design was re-assessed by various investigators. It was recognised with the increase in size of these steel structures, and the need for the best economic utilization of materials, that a rational method of structural design, correlated with soil mechanics was required. In 1941 M. G. Spangler(24) developed a rational method for determining the deflection of steel pipe. In 1957 H. L. White suggested the use of ring compression theory.(32) Also investigations were carried out by the Prairie Farm Rehabilitation Administration in Canada and by others in the United States to assess the behavior of the soil around the flexible steel conduits. In 1961 a research project was conducted at Nova-Scotia

Technical College sponsored by the corrugated steel pipe industry. Scale models of 90 degree arch segments bearing against a sand fill were studied under an axial load. Results of this work including a method of ring stress analysis governed by buckling was published in 1963 by G. G. Meyerhof and L. D. Baikie (17).

The interaction between a flexible metal culvert structure and the surrounding backfill was studied by Duncan & Clough (1971), Bjerrum, et al (1971), Allgood & Takahashi (1972), Abel, et al (1974) by using a finite element approach. In this finite element analysis the nonlinear and stress dependent stress-strain behavior of the backfill and the actual sequence of construction operations were taken into account.

In 1972 laboratory model tests were conducted by A. K. Howard and in 1976 same author suggested values for the modulus of subgrade reaction for various types of soil, which is an important parameter to be used in the Iowa Formula (12).

In 1978, Abdel Sayed (2) studied the stability of the culvert wall and suggested formulas for calculating the elastic and inelastic buckling stresses.

In the analysis of the live load effects on soil-steel structures, the finite element analysis can be used to realistically model the conduit in three dimensions. This analysis is expensive since it requires considerable computer size and time. A 'Plane-strain' analysis is used to reduce the three dimensional analysis to two dimensions by assuming that there are no variation in the force effects in the longitudinal direction of the conduit. The model is realistic when applied to conduit with dead loads or line loads, but does not accurately predict effects due to the concentrated live loads especially under shallow covers.(1). Abdel Sayed and Bakht have stated that the live load forces effects form a small part of the total force effects , especially in a structure with large depths of cover(1) but in the case of shallow cover, the effect of live load is a considerable part of the total force effects and assumption of 'plane-strain' may not be accurate.

All the above studies consider no variation of behavior of the conduit along its axis (i.e. longitudinal direction)

1.3 Objectives

Generally the soil steel structures have the ratio of height of soil cover to the span, greater than 1/6 . In the case of shallow covers the force effects due to the external live load vary in the longitudinal direction and the loads on such structures can not be resisted entirely through the ring compression action.

The objectives of this investigation are

(i) To study the live load dispersion pattern through a soil media (the three dimension effect of load) and suggest a rational way to determine the live load dispersion through the soil media.

(ii) To check the validity of the 'Ontario Highway Bridge Design Code' provisions to estimate maximum unit thrust (lb/inch) induced in a conduit due to the live load under the condition of shallow cover.

(iii) To study the bending moment distribution in a conduit due to the live load.

(iv) To study the deflection of conduit due to the live load.

1.4 Scope

Two conduit models under the shallow backfill were tested under the action of concentrated loads. From the test data the live load dispersion in the soil is determined, an expression is proposed for calculating the thrust in the conduit wall and the deflection formulas of soil-steel structure are checked.

CHAPTER II

EXPERIMENTAL INVESTIGATION

2.1 Scope Of Experimental Program

The objectives of this experimental investigation are to measure the thrust, bending moment and soil pressure at specified locations along the circumferential and axial direction of the conduit. Measurement of deflections of the conduit wall along the midsection is also part of this investigation.

The experimental program consisted of tests on two circular aluminium conduit models, Fig. 2.1 , 2.4 and 2.5. One was relatively rigid and the other was relatively flexible. Conduit#1 was tested for three different soil covers over the crown, three incremental point loads and three eccentricities of the load. Conduit#2 was tested for four different soil covers over the crown, three incremental point loads and three eccentricities of the load. The unit strains at inner and outer fibers of the conduit wall at various locations (Fig. 2.2 & 2.3) were measured. Pressure variation along the length of conduits, at the crown level and along the circumference of the conduits were recorded for various magnitudes of loads and backfills by means of pressure cells (Fig.2.2 & 2.3). The deflections of the conduit wall were recorded by means of deflection gages installed at various locations in the radial directions (Fig. 2.6).

2.2 Model Study Of Soil-Steel Structure (18)

Model tests have been successfully used to investigate the performance of the full scale culverts. A model test is a powerful tool in the investigation of complex problems that can not be readily evaluated by using analytical means. The use of models in obtaining solutions related to the performance of underground structure is the most attractive because of the complexity of the problems involved. Time and money often prohibit the use of full scale tests to investigate the effect of different variables on a system. Due to the ease with which models can be fabricated and tested, they yield much more information for a given amount of time and money than do full scale tests. Full scale tests are useful in verifying the results obtained from model tests.

In order to establish reliable similitude requirements for a given model system, all variables that influence the phenomena must be defined.

2.2.1 Similitude Requirements

In the design of culverts the Iowa formula is widely used. Therefore this formula will be used to demonstrate the modelling concept.

Change in the horizontal diameter or the vertical deflection of the culvert is given by (24)

$$\Delta = D_l \cdot K \cdot \frac{WR^3}{EI + 0{\cdot}061\, E'R^3} \qquad \text{--------- Eq. 2.1}$$

In which Dl:- deflection lag facter to compensate for the volume change of the soil with time, dimensionless

K= bedding constant which varies with the angle of the bedding, dimensionless.

W= load on the pipe per unit length, in pounds per linear inch.

=p*2R

p= pressure in pounds per square inch at the crown level.

R= pipe radius, in inches =D/2

D=diameter of the pipe, inches

EI= pipe wall stiffness per inch length, inch-pound.

E'= modulus of soil reaction, in lb/sq.in.

Rearranging the terms of the Iowa formula

$$\frac{\Delta}{D} = D_l \cdot K \cdot \frac{pR^3/EI}{[1+0.061E'R^3/EI]}$$

It is observed that the terms DEL/D, pR^3/EI and $E'R^3/EI$ are the nondimensional numbers.

i.e. $$\pi_1 = D_l \cdot K \cdot \frac{\pi_2}{(1+0.061 \cdot \pi_3)}$$

Consider the following average prototype

D= Diameter= 26 ft.= 312 inches.

E=Modulus of Elasticity= 30×10^6 psi. (Steel)

5 Gage, 6 inch.x 2 inch. corrugations.

I=Moment of Inertia= 0.113 in^4/in.

p=Equivalent pressure at the crown level, say 5.0 psi.

E'=Modulus of Soil Reaction, say 4000 psi.

Therefore the nondimensional terms for this prototype are

$$\pi_2 = P \cdot R^3/EI = 5 \times 156^3/(30 \times 10^6 \times 0.113) = 5.6$$

$$\pi_3 = E'R^3/EI = 4000 \times 156^3/(30 \times 10^6 \times 0.113)$$

$$= 4479.6$$

Therefore π_1 =DEL/D=0.0017 i.e. 0.17 %

Now to arrive at a model size to represent the above average prototype in field the following assumptions are made.

D=diameter of the model to be used = 30 inches

E=modulus of elasticity of a model material

$=10 \times 10^6$ psi (made of aluminium)

p=equivalent pressure at the crown level=2.5 psi.

and E'=modulus of soil reaction = 2000 psi.

To have the same percentage horizontal deflection [i.e. π_1 =0.17 %], on substitution of above parameters in the Iowa equation, we get the thickness of the conduit wall equal to 3/16 inch.

Thus a model of thickness 3/16 inch has the same flexibility and performs in a similar way as the prototype conduit. A second conduit of thickness equal to 5/16 inch is also tested in order to investigate the effect of increasing the rigidity of the conduits on the performance.

2.3 Materials

2.3.1 Soil

The soil used in this experimentation was clean, dry sand from Lake Erie. The bulk density of the sand was found to be 116 pcf. On compaction the density of this sand did not increase significantly and was equal to 119 pcf.(see Appendix C)

The value of modulus of soil reaction E' for this type of sand was suggested in the range of 1000 psi-3000 psi (12). For a medium compacted sand, the modulus of soil reaction (E') of 2000 psi has been used in the present calculations.

2.3.2 Conduit

Conduits were fabricated from aluminium alloy 6061-T-6 (Extruded) for which modulus of elastisity (E), yield stress (σy) and poisson's ratio (m) were $10x10^6$ psi, 40 ksi and 0.33 respectively.

2.4 Description Of The Test Models

The dimensional details of the conduit models were as shown in Fig.2.1 and the properties as listed below.

Conduit#1 ----- span (diameter)=30.0 inch.
thickness =5/16 inch.
moment of inertia=1/393 in^4/in.
length =60 inch.

Conduit#2------ span (diameter) =31.0 inch.
thickness =3/16 inch.
moment of inertia=1/1820 in^4/in.
length =60 inch.

The conduits were fabricated from standard 48 inch. wide aluminium plates. The 60 inches length of conduit was achieved by welding an additional aluminium ring 12 inch. wide to the 48 inches long conduit. The longitudinal joints of 48 inches wide piece and 12 inches piece were staggered to have a strong joint. Circular holes of 4 inches diameter were cut out from the conduits at desired locations (Fig.2.2 & 2.3) to mount the pressure cells by a disk type power drill.

2.5 Instrumentation

In this investigation measurements of pressure, strains and deflections were needed. Therefore to measure these parameters, diaphragm type pressure cells, electrical resistance strain gages and deflection dial gages were used. Also to apply a specified magnitude of concentrated load to conduit model, a Strainsert flat universal type load cell was used to monitor the load.

2.5.1 Diaphragm Type Pressure Cells(15)

The construction procedure of the diaphragm type pressure cells is described in reference(15) and the details are furnished in fig.2.7.

Each pressure cell was calibrated using a calibration device as shown in figure 2.8 . The air pressure of the main line of 100 psi was reduced to 30 psi by means of a regulator. The air pressure was applied to the top of sand in an air tight container through one psi increament pressure values and corresponding strain indicator readings experienced by the pressure cell were recorded. The calibration charts for all pressure cells are presented in Appendix A.

2.5.2 Electrical Resistance Strain Gages

The strain gages used were of the type CEA-13-500UW-120 with gage factor of 2.09 +/- 0.5% at 75° F and transverse sensitivity factor 'Kt' equal to -0.2 %. The strain gage locations on the conduit were sanded to make the surface smooth (but not extremely smooth). The surface was then cleaned with acetone and conditioned by conditioner solution, which was then neutralized by neutralizer solution. The gages were positioned by means of a rigid transparent tape. The position and the orientation of the gages were maintained by the tape as the adhesive (M-200 bond) was applied and as the gages were pressed into place for about three minutes by squeezing out the excess adhesive. After testing, the gages were covered by a protective layer of M-coat. In this fashion 44 (22 outside & 22 inside) strain gages were installed for each test conduit. It was carefully observed that nowhere were the strain gages located at a distance less than four times the radius of cutout hole from the centre of the cutout to prevent the strain gage readings getting affected by the stress concentration around the hole (fig.2.2 & 2.3). The strain gages installed on the outer surface of the conduits were protected from the effects of soil pressure by means of a U-shaped 0.3 inch thick rubber piece with a thin aluminium sheet pasted on its top, creating a hollow space for the strain gage. This strain gage assembly was then covered by a "durable gray" tape to secure the protective cover. (fig. 2.13). The strain gages were connected to the multichannel automatic digital strain indicator. (fig.2.9)

2.5.3 Deflection Gages

One inch range deflection gages were used to measure the deflection of the conduits along the radial direction. The dial gages were installed on a frame as shown in fig. 2.6. To make the dial gage readings clearly visible a 40 W lamp was used to focus a light on the dial gages.

2.5.4 Load Cell

A load cell (Strainsert flat universal type) was placed on the top of the loading plate and below the hydraulic jack (fig.2.10) to measure the magnitude of the applied concentrated load. This load cell with a maximum load capacity of 10,000 lb. was calibrated with a portable strain indicator (Budd Model P-350).

2.6 Experimental Set Up And Test Procedure

A rectangular box shaped test cell of size 12 ft. long x 5 ft. 1 in. wide x6 ft. high was constructed by means of 3/4 in. plywood, 1/2 in. thick plexiglass and the structural angles as shown in fig.2.11. A soil bed 12 inches deep was laid and compacted manually in two layers to achieve a soil density of 119 pcf.(fig.2.12). The conduit model with strain gages and pressure cells installed on it, was then laid on this prepared bed. Deflection dial gages were supported on a frame inside the conduit (fig.2.6). The strain gages and the pressure cells were connected to the automatic digital strain indicator box (by which pressure cells were calibrated) respectively. Before the laying of any soil layer above the initial bed level, the strain gage and the pressure cell readings were adjusted to zero with the proper gage factor set. The conduit ends were wrapped with foam rubber to seal any gap between the conduit end and the rectangular test cell. It was carefully observed that the sealing of the gap did not obstruct the free movement of the conduit wall at the ends. The pressure cells (with the sensitive membranes exposed to the soil) were covered by means of a thin rubber membrane to avoid any rusting of the pressure cell's membrane. The soil on both sides of the conduit was placed in 7.5 inches layers and compacted evenly to achieve a soil density of 116-119 pcf; and strain, pressure & deflection readings were recorded. This procedure was repeated until the soil cover over the crown of the conduit became 5 inches.

The actual test started when the loading plate was centered at the section A-A of the conduit with 5 inches of backfill over its crown. The longer size of the loading plate was adjusted parallel to the span of the conduit. The load cell was then centered on the loading plate. With this set up the initial zero live load strains, pressures and deflections were recorded. The load of 1,000 lb. was then applied on the loading plate and was kept constant until all the readings were recorded. Similarly, readings were taken for higher loads of 1,500 lb. & 2,000 lb. at the same point. For each loading point the initial and final zero load readings were also recorded. The position of the loading point was then changed to an eccentricity of 7.5 inches from the centre of the conduit and similar sequence of recording the readings was followed. Finally the loads were applied to the point at an eccentricity equal to 15 inches from the centre of the conduit and the strain, pressure & deflections were recorded. In the same way loads were applied at sections B-B and C-C with eccentricities of zero to 15 inches. The entire procedure of test was repeated for the soil covers of 7.5 inches and 10 inches over the crown. At the end of the test of 10 inches cover the test cell was emptied to the bed level and the conduit#1 was removed.

In a similar fashion conduit#2 was placed, buried and tested. In this case the sequence of loading points was slightly altered just for convenience in conducting the test. The loads for each soil cover were applied at sections A-A, B-B and then at C-C with zero eccentricity and afterwards loads at 7.5 inches and 15 inches eccentricities were applied for sections A-A, B-B and C-C (Appendix-B, K-Identification Number). Conduit#2 was tested for four soil covers from 5 inches to 12.5 inches through 2.5 inches increaments.

For each load increament the settlement of the loading plate was recorded by the deflection dial gage.(fig.2.10)

2.7 Symbolic Representation Of The Loading Conditions.

The loading conditions are represented by symbols Ai(L)-H, Bi(L)-H, Ci(L)-H, where alphabets A, B, C indicate the loading sections AA, BB and CC respectively. Subscript 'i' indicates the eccentricity of the load

i.e. i=1 represents e=0. inch.

i=2 represents e=7.5 inch.

i=3 represents e=15. inch.

The bracketed quantity (L) indicates the magnitude of concentrated load (0 to 2,000 lbs). The term 'H' indicates the magnitude of the soil cover over the crown. For example - a load of 1,500 lb. at a section CC with zero eccentricity and soil cover of 7.5 inches is represented by the symbol C1(1,500)-7.5".

The moments, thrust and pressure are represented by the symbols M(K,L,I), T(K,L,I) and P(K,L,I) respectively where K-represents the position and eccentricity of the load

and the magnitude of the soil cover. (Appendix B)

L-represents the magnitude of the load.

e.g. L=1 represents P=0 lb. (Initial)

L=2 represents P=1,000 lb.

L=3 represents P=1,500 lb.

L=4 represents P=2,000 lb.

L=5 represents P=0 lb. (final)

I represents the strain gage location (1 to 22 corresponding to the strain gages on the outer surface of the conduit) for thrust and moment or pressure cell location (I=1,10) for pressure. (fig.2.2 & 2.3)

2.8 Interpretation Of The Test Data

2.8.1 Calculation of load dispersion angles Alpha and Beta

The actual size of the loading plate was 8 in. x 11 in. The longer size of the loading plate was kept perpendicular to the length of conduit. The distribution of the reactions of soil below the loading plate depends on the condition of soil at the boundary of the loaded plate.

For the similar condition as in the present test the ratio of the maximum pressure below the plate and the average pressure (P/A) is assessed to be 2.5 as shown in fig.2.14. (Ref. 26, page 209)

For calculation purposes the pressure below the loading plate is assumed to be uniform in order to simiulte the pressure under a truck wheel and consequently the size of the loading plate reduced to an equivalent size which would give the assumed uniform pressure below it. Thus the equivalent (reduced) area of the loading plate is obtained by dividing the actual area of the plate by 2.5. In other words the equivalent sides of the loading plate are obtained by dividing the actual side by $\sqrt{2.5}$

i.e we=equivalent width of loading plate

=actual width x $1/\sqrt{2.5}$ =$11/\sqrt{2.5}$= 7.0 inches.

be=equivalent length of loading plate.

=actual length x $1/\sqrt{2.5}$ = $8/\sqrt{2.5}$ =5.0 inches.

The measured values of the pressure on the top and along the length of conduit were plotted for each load. (fig.2.15 to 2.21)

The actual pressure distribution on the conduit is very difficult to be considered in the practical analysis of a soil-steel structure. Therefore, designers consider an imaginary angle of dispersion alpha(α) in the longitudinal direction and beta(β) in the span direction. (ref. 3 and 21, Fig.2.22) This leads to considering an imaginary effective area (2bc x 2wc) on which the pressure is assumed to be constant and equal to the maximum pressure qc.

The equivalent width of dispersion (2bc) along the length of the conduit is calculated by dividing the area of the pressure envelope by maximum pressure recorded on the top of the conduit for a given load.

The angle of load dispersion (α) in the direction of the length of conduit was worked out by equation 2.2

Alpha=tan-1[(2bc-be)/2H*]....................Eq. 2.2

Where H* is the actual depth of cover on the top of conduit.

H*=H-settlement of the loading plate

=H-0.30" for P=1,000 lb.

=H-0.45" for P=1,500 lb.

=H-0.60" for P=2,000 lb.

The equivalent width of the load dispersion on the top of the conduit in the span direction (2wc) is calculated as follows

$$2w_c = (P\_actual / 2b_c) \times 1/2 \text{ ----------------- Eq. 2.3}$$

where, P_actual- is the applied load on the plate.

qc - is the measured soil pressure on the crown level under a concentrated load 'P_actual' acting at a section A with zero eccentricity.

The load dispersion angle (β) in the direction of span of conduit is calculated as follows

$$\beta = \tan^{-1}[(2w_c - w_e^{*})/2H^*] \text{ ----------------- Eq. 2.4}$$

The calculations are shown in table 2.1 and the values of Alpha and Beta are tabulated in table 2.2 for both conduits under different soil covers.

2.8.2 Calculation Of Maximum Thrust And Bending Moment

The strain readings were split into the strains due to direct thrust and the strains due to the bending moment as shown in fig.2.23, where:

Eav=Average unit strain=(Eout+Ein)x1/2.........Eq. 2.5

=strain reading due to the axial thrust only.

Ebend=strain due to the bending moment.

= Ein - Eav ....Eq. 2.6

where a +ve value of 'Ebend' indicates tension on the inner fiber.

Unit axial thrust $T = \frac{E}{(1-m^2)} \times E_{av} \times 10^{-6} \times t \times 1$ .... Eq. 2.7

Bending Stress= $\sigma_{bend} = \frac{E}{(1-m^2)} \times E_{bend} \times 10^{-6}$ ....Eq. 2.8

Bending Moment M= $\sigma_{bend} \times I/c$ ....Eq. 2.9

In this way all strain gage readings at various locations, were converted to the unit axial thrust and moment values. The experimental maximum thrust values are presented in Table 2.3. The experimental moments at crown and θ=36 deg. location are also presented in tables 2.4 and table 2.5 respectively.

CHAPTER III

DISCUSSION OF RESULTS

3.1 General

In this work though the conduits were tested for both axial and eccentric loads the results are presented here only for the load at midsection A-A (axial load) with zero eccentricity since this position of load yields the most critical stress conditions in a conduit.

3.2 Load Dispersion Angles Alpha & Beta.

The angles alpha and beta (Table 2.2) are found to be varying for different soil covers. The angle alpha in the longitudinal direction decreases as the backfill over the crown increases. But the reverse is the trend found in the angle beta for the span direction. Beta increases with increase in the backfill over the crown.

It is also observed from Table 2.2 that the values of alpha and beta also depend on the flexibility of the conduit. For conduit#1 the angle alpha decreases from 27 degrees to 23 degrees as the cover increases from 5 inches to 10 inches. The same angle for conduit#2 varies from 24 degrees to 16 degrees for increase in soil cover from 5 inches to 10 inches. The angle beta has an opposite trend. It increases with increase in the soil cover. The change/increase in angle beta for conduit#2 at 10 inches. cover is sudden, while for conduit#1 it is gradual and uniform for the tested cover heights. The sudden jump in the angle beta at 10 inches cover for conduit#2 can best be explained by the soil arching phenomenon. Once the cover reaches a value of 10 inches, for conduit#2, a considerable part of the live load is taken up by the soil itself and a small portion of load is carried by the conduit. With the increase in flexibility of the conduit the height of backfill required to develop the soil arching action is reduced. For conduit#1 this soil arching action was not observed to take place for the range of soil covers tested.

Theoretically the load dispersion angles alpha in the longitudinal direction for rigid conduits (e.g. conduit#1) are expected to be smaller than those for flexible conduits (e.g. conduit#2) under similar conditions, since a concentration of stresses in the rigid conduit are expected. But the experimental alpha values contradict this view point. The measured alpha values for conduit#1 are higher than those for conduit#2. The load dispersion angle beta in the span direction could have an influence on the values of alpha.

3.3 Proposed Thrust Equation

Consider the equilibrium of an element of the conduit (fig. 3.1a) and by neglecting the forces in the X direction (i.e. longitudinal direction) we get the equilibrium condition for this element as follows (16):

$$RN_\theta + \frac{\partial^2 M_x}{\partial \theta^2} + R.P_z = 0 \qquad \text{Eq. 3.1}$$

Dividing equation by R and rearranging we get

$$N_\theta = -[R.P_z + \frac{1}{R} \times \frac{\partial^2 M_x}{\partial \theta^2}] \qquad \text{Eq. 3.2}$$

The experimental thrust values at various angular locations in the conduit wall show that their vertical components equal approximately the shear force at that location, when neglecting the bending moments in the conduit. With this observation, we can therefore say that the contribution of the term $\frac{1}{R}\times\frac{\partial^2 M_x}{\partial\theta^2}$ in equation 3.2 is almost negligible. Therefore equation 3.2 can be written approximately as follows,

$$N\theta = -R.P_z \qquad \text{Eq.3.3}$$

This equation for thrust ($N\theta$) is applicable in the region of conduit covered by the loading strip Pz. In figure 3.1b the free body diagram of the conduit wall is shown by neglecting the bending moments, since these moments do not contribute appreciably to axial thrust in the case of flexible culverts. With this approximation the axial thrust for the upper half portion of the conduit, is given by equations 3, 4 and 5 of fig.3.1b. Thus equations 3 & 5 of fig. 3.1b and equation 3.3 are identical.

The axial thrust in a portion of the conduit not covered by the equivalent loading width is given by equation 4 of fig 3.1b i.e.

$$T = P_{max}.wc/\sin\theta \qquad \text{for } \sin^{-1}(wc/R) \leqslant \theta \leqslant 90 \text{ degrees}$$

If the equivalent dispersion width 2wc is greater than the span of the conduit, the proposed equation gives a constant thrust of

$$T = P_{max}.R \qquad \text{for } 0 \leqslant \theta \leqslant 90 \text{ degrees}$$

which is same as equation 3.3 derived from the equilibrium concept.

This proposed thrust equation gives the maximum and constant value of thrust in a conduit portion covered by the equivalent live load dispersion width 2wc.

The maximum thrust values obtained by the proposed thrust equation are presented in the table 3.1.

The variation of maximum thrust with respect to the soil cover and the live load are plotted in fig. 3.2 and fig. 3.3

Fig. 3.4 and fig. 3.5 show that the experimental maximum thrust occurs at the crown level for soil cover upto 7.5 inches while it appears to be shifting to the point B (at radial location θ=36°) for cover greater than 7.5 inches.

It is observed from the table 3.1 that the maximum thrust decreases with the increase in soil cover. This is because the intensity of load pressure Pz on the crown level decreases with increase in the soil cover.

The maximum thrust values for conduit#1 are observed to be smaller than those for conduit#2 under similar conditions. This is due to the fact that conduit#1 was relatively rigid compared to conduit#2. In the case of rigid conduits the flexural stiffness helps resist the external loads while for flexible conduit the flexural stiffness was small and hence the load was resisted mainly by the axial thrust.

3.4 Bending Moments In The Conduit.

It is observed that the maximum bending moment occurs at the crown of the conduit under a concentrated load acting at midsection with zero eccentricity (fig. 3.6 and 3.7). The moment at the crown causes tension on the inner fiber. The point of contraflexure is observed to be at approximately θ=30 deg. The maximum negative moment (causing tension on the outer fiber) occurs at approximately θ=36 degrees location. And for the rest of the conduit the bending moment is almost negligible. This is true for both flexible and rigid conduits.

From the tables 2.4 and 2.5 it is observed that the bending moments in conduit#1 are much higher than those in conduit#2 under similar condition. This is due to the fact that conduit#1 has more flexural rigidity.

The variation of the bending moments at A (crown level) and at B (θ=36°) are plotted in fig. 3.8 through 3.11. If these curves are extended to cut the vertical axis i.e. the soil cover axis, all the curves intersect in one point. This common point on the vertical axis gives important information regarding the minimum depth of soil cover over the crown required to make the conduit free of the bending moments. The bending moments at A for conduit#1 and conduit#2 vanish when the soil cover reaches a value of 14.5 inches and 13 inches respectively The bending moments at B for conduit#1 and conduit#2 vanish when the cover reaches a value of 17.5 inches and 13.8 inches respectively. Thus the soil cover required to vanish the bending moments in conduit#1 is higher than that required for conduit#2. Thus minimum h/D ratio for conduit#1 and conduit#2 to vanish the moment at A are 0.48 and 0.42 respectively while that for conduit#1 and conduit#2 to vanish the moment at B are 0.58 and 0.45 respectively. In general the minimum h/D required for rigid conduit is higher than that for flexible conduit.

Theoretically it was expected that the values of h/D required to vanish moments at A and B should be the same. When the soil cover becomes equal to the specified minimum value, the pressure distribution around the conduit is uniform. This is similar to a ring subjected to the uniform radial pressure, thereby subjected to pure ring compression. If the soil cover is less than the specified minimum value, radial pressure acting at the midsection is no more uniform hence the conduit (or ring with non-uniform radial pressure) experiences thrust along with the bending moments.

Now the higher value of h/D required for conduit#1 can be explained as follows:

The radial pressure around the conduit approaches the uniform value as the conduit deflects more. The deflection of conduit#1 is less than the deflection of conduit#2 under similar conditions. Hence the radial pressure for conduit#2 becomes uniform at smaller cover height, while for conduit#1, it becomes uniform at higher cover height.

3.5 Deflection Of The Conduit

Since the measured deflections for conduit#1 were very small for the range of soil covers tested, these are not considered here. But those for conduit#2 are sufficiently large and are studied here and presented in table 3.2. The deflections (Δx) predicted by the Spangler's equation are calculated and compared with the measured deflections. The value of the modulus of soil reaction (E') considered in the calculations for the type of sand used in the experimentation was assessed to be equal to 2,000 psi.(12). Also it was felt to be neccessary to introduce the effect of shallow cover by modifying the value of modulus of soil reaction (E'). First modification considered here is (14)

$Em' = E'[1 - \{R/(R+h)\}^2]$ while the second modification considered is

$Em' = E'[1 - \{R/(R+h)\}^3]$

The deflections for conduit#2 are calculated by taking into account the above two modified values of soil reaction (Em'). The value of Wc term in the Spangler's equation was calculated by assuming the load dispersion specified by OHBD code (19). The deflections calculated by the original Spangler's equation (23,24) and by using the modified values of E' for the values of modulus of soil reaction in the range of 1,000 to 3,000 psi. are presented in Appendix B. Observation of these results indicate that the deflection predicted by the original Spangler's equation are slightly underestimated for smaller cover values while the values obtained by the first modification(14) are very much on conservative side. However the second modification predicts higher but less conservative deflection values as compared to the measured values. If the measured deflection values are equated to the above three equations, the calculated values of E' are found to be approximately 1,900 psi., 3,000 psi. and 2,100 psi. for the original equation, first modification and second modification respectively. The modulus of soil reaction value for the sand used in this experimentation was assessed to be equal to 2,000 psi. for medium compacted sand (12), which means that the deflections predicted by second modification are more on the acceptable side for soil covers less than 7.5 inch. The variation of deflections with respect to the height of cover for given load are presented in fig.3.12.

CHAPTER IV

EVALUATION OF VARIOUS METHODS OF DESIGN.

4.1 Introduction

From the results of this investigation the load dispersion angles (α and β) for conduit#1 and conduit#2 for various soil covers over the crown level are suggested and by making use of this load dispersion pattern a thrust equation has been proposed. Also on the basis of measured deflection at the crown level of conduit#2 a modification has been studied in the Spangler's equation. The measured thrusts and proposed thrust values are compared with those predicted by OHBDC(19) and AASHTO(3). In the following section these methods (proposed, OHBDC, AASHTO and experimental) are evaluated.

4.2 Load Dispersion Angles (Alpha & Beta)

OHBDC and AASHTO codes specify constant values of alpha and beta for any magnitude of soil cover (fig 2.22). Experimentally it is observed that these values vary not only with the magnitude of the soil cover but also with the flexibility of the conduit. In practice the soil-steel structures are always flexible with flexibility factor, D^2/EI, fairly constant (which ranges from 2x10-2 in./lb. to 10x10-2 in./lb.). Therefore the code of practise specifying the load dispersion angle without taking into account the flexibility factor is understandable. But the height of backfill does change from shallow to deep.

Therefore variable values of alpha and beta must be specified for various values of backfill. AASHTO does not make any difference between the load dispersion angles in the span and longitudinal directions, while OHBDC specifies different load dispersion angles in these directions. Experimental values of load dispersion angles indicate that these are not the same in the longitudinal and span direction. Thus the codes of practise do not make use of soil arching action in the design. The inclusion of varying alpha and beta values takes into account the soil arching phenomenon.

4.3 Axial Thrust

The axial thrust calculated by OHBDC(19) and AASHTO (3) are tabulated in table 4.1 and 4.2 The measured thrusts, thrusts by the proposed equation, OHBDC and AASHTO are plotted in figure 4.1 through 4.6 for conduit#1 and conduit#2. It is observed from these figures that the OHBDC thrust values(smaller of T-1 and T-2) for shallow cover are very much underestimated while the AASHTO thrust values are less underestimated. The proposed thrust equation making use of varying alpha and beta values estimates the thrust very close to the measured values. Thrust T-1 (=p.r) by OHBDC is less underestimated.

Both the codes give constant value of thrust in a conduit ring while the measured values of the thrust indicate that the maximum thrust occur inbetween crown and θ=36 degrees location and this thrust goes on decreasing for the remaining portion of the conduit and becomes almost zero at the bottom of the conduit. The proposed thrust equation gives a constant value of thrust over a portion of the conduit covered by the equivalent width of the load dispersion (2wc) at the crown level and this value decreases beyond the equivalent dispersion width, Fig.4.7 . The proposed equation is applicable to the top half of the conduit (i.e. up to the spring line), which is sufficient from the design point of view since the thrust in the bottom of the conduit is not critical.

4.4 Minimum Height Of Soil Cover

If the cover over the conduit crown is below a certain minimum value, the live load causes high bending moments in a conduit. This bending moment is very high at the crown of the conduit, causing tension on the inner fiber, when the load is acting at AA with zero eccentricity. The maximum negative moment (that causes tension on outside fiber) occurs at B, i.e. at θ=36 degrees location.

The minimum value of h/D for conduit#1 required to make the conduit free of bending moments ranges from 0.48 to 0.58 while the same value for conduit#2 lies approximately between 0.42 and 0.45. OHBDC(19) specifies minimum value of height of backfill equal to 1/6th span of the conduit, i.e. the minimum h/D=0.17. The AASHTO(3) specifications specify this minimum value equal to the smaller of 12 inches or span/15. Thus (h/D)min. by AASHTO lies between (31/12x15=) 0.17 and (12/31=) 0.39. The flexibility factor for conduit#1 was equal to 3.54x10-2 in./lb. while that for the conduit#2 was equal to 17.5x10-2 in./lb. The flexibility factor for the practical culvert lies between 2x10-2 in./lb. and 10x10-2 in./lb. (ref. 3,19). Therefore the minimum h/D ratios obtained for the test models are the most practical values. Thus the specified value of h/D ratio by the codes are small and hence in the actual design with prevailing h/D limitation, the bending moments must be taken into account or else the h/D ratio should be taken higher.

4.5 Deflection Of The Conduit

The Spangler's equation(23,24), which is widely used to predict the deflection of the culvert predicts a small value of deflection for culverts with h/D less than or equal to 1/4. The same equation with the third order reduction in the value of modulus of soil reaction predicts fairly accurate deflection for h/D $\leqslant$ 1/4. But the original Spangler's equation for soil cover greater than 7.5 inch (i.e. h/D > 1/4) gives acceptable values of deflection.

CHAPTER-V

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The overall objective of this study was to conduct a laboratory investigation of a 'soil-steel structure' with shallow cover subjected to a concentrated load to study (i) the live load dispersion pattern through soil media, (ii) the thrust and bending moment in such structures, (iii) deflection at the crown level and (iv) to check the validity of the OHBD code with regards to the above points. Therefore tests were carried on two conduit models. The experimental results led to the following conclusions:

(i) The angles of load dispersion in the longitudinal and the span directions are different and vary with the depth of soil cover and the flexibility of the conduit. However their magnitudes can be approximated as given by the OHBD code.

The angle alpha decreases and the angle beta increases with increase in the depth of soil cover.

(ii) In the case of conduit#2, the angle beta jumps suddenly from 24 degrees to 55 degrees at 10 inches cover or at h/D=1/3, indicating the development of an arching action at h/D=1/3 meaning that the major portion of the load was transfered to the adjoining soil mass.

For conduit#1 the angle beta did not show sudden change.The change in beta seems to be gradual and uniform with respect to the depth of soil cover.

(iii) The pressures at the crown level of the conduit estimated by the code provisions are smaller than the measured values (Table 4.4).

(iv) Unlike the OHBD and AASHTO codes, the axial thrust is not constant in a conduit wall but is observed maximum at the crown and zero at the bottom. The constant thrust is possible only when the soil cover over the crown is greater than the minimum depth of soil cover required to vanish the bending moments in a conduit.

(v) The maximum thrust (smaller of p.r and P'/2) estimated by the OHBD code is very small for the condition of shallow cover while this thrust by AASHTO specifications is less underestimated. The proposed equation gives more or less the same thrust values as that measured.

(vi) The minimum h/D ratio required to make the conduit free from the bending moments is 0.42 while the same ratio specified by the OHBDC is 1/6 and by AASHTO 0.38 which are smaller than the measured h/D value.

(vii) The original Spangler's equation underestimates the deflection values for h/D less than 1/4. (i.e. for the condition of shallow cover.)

5.2 Recommendations

Following are the recommendations suggested on the basis of this experimental investigation:

(i) In the calculation of the equivalent pressure at the crown level due to the live load, the use of varying angles of load dispersion (alpha and beta) with respect to the height of backfill and the flexibility factor of the conduits should be made.

(ii) The thrust in a conduit should be estimated by the following expressions:

T=Pmax.R for the portion of the conduit covered by 2wc.

or T=Pmax.w/sinθ for w $\geqslant$ wc and $\leqslant$ R

or θ $\geqslant$ sin-1(wc/R) & $\leqslant$ 90°

(iii) The minimum h/D ratio should be 0.42; if less than this value, the bending moments should be considered in the design.

(iv) The modulus of soil reaction E' should be modified to $E'[1-\{R/(R+h)\}^3]$ to predict the deflection of the conduit for h/D less than or equal to 1/4.

(v) One more conduit with corrugations or circumferntial stiffeners to represent the orthotropic nature of practical 'Soil-Steel Structures' should be tested.

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TABLES

CONDUIT NO	SOIL COVER H" Inch	LOAD P-1b.	q_c Psi	$A_p = \int f(x) \cdot dx$	$2b_c = A_p/q_c$	$\alpha = \tan^{-1}\left\{\frac{2b_c - 5}{2H''}\right\}$	α_{av}	$2w_c = \frac{P}{2b_c} \cdot \frac{1}{q_c}$	$\theta = \tan^{-1}\left\{\frac{2w_c - 7}{2H''}\right\}$	θ_{av}
1	4.70	1000	7.20	66.0	9.17	24.0	27.1	15.2	41.0	27.6
	4.55	1500	13.20	128.0	9.70	27.3		11.7	27.3	
	4.40	2000	21.60	216.0	10.00	30.0		9.3	14.6	
	7.20	1000	4.20	50.0	11.90	26.0	27.5	20.0	42.0	34.2
	7.05	1500	7.20	88.0	12.20	27.0		17.0	35.3	
	6.90	2000	11.60	149.0	12.80	29.5		13.5	25.2	
	9.70	1000	2.80	30.0	12.24	20.5	22.6	29.2	49.0	42.0
	9.55	1500	5.15	58.0	13.80	24.7		21.1	36.4	
	9.40	2000	6.75	80.0	12.80	22.5		23.2	40.8	
2	4.70	1000	10.40	78.1	7.51	15.0	23.7	12.8	31.7	23.6
	4.55	1500	15.00	138.3	9.22	25.0		10.9	23.2	
	4.40	2000	20.40	210.5	10.32	31.2		9.5	16.0	
	7.20	1000	6.20	61.0	9.84	18.6	20.3	16.4	33.1	31.2
	7.05	1500	9.20	95.2	10.34	20.7		15.8	32.0	
	6.90	2000	13.20	138.0	10.45	21.6		14.5	28.5	
	9.70	1000	2.45	24.0	9.80	14.0	15.6	41.7	60.8	55.4
	9.55	1500	4.20	43.6	10.38	15.7		34.4	55.1	
	9.40	2000	6.25	67.6	10.81	17.2		29.6	50.2	
	12.20	1000	2.05	22.0	10.76	13.4	13.0	45.5	57.6	57.5
	12.05	1500	3.13	33.0	10.53	13.0		45.5	58.0	
	11.90	2000	4.54	46.0	10.13	12.2		43.5	57.0	

TABLE 2.1 CALCULATION OF LIVE LOAD DISPERSION ANGLES FROM EXPERIMENTAL DATA

SOIL COVER H - INCH	ANGLES OF LOAD DISPERSION FOR			
	CONDUIT # 1		CONDUIT # 2	
	ALPHA(α)	BETA(β)	ALPHA(α)	BETA(β)
5	27	28	24	24
7.5	27	34	20	31
10	23	42	16	55
12.5	-	-	13	58

TABLE 2.2 LIVE LOAD DISPERSION ANGLES - BASED ON PRESSURE READING AT CROWN LEVEL

CONDUIT NO.	SOIL COVER H - Inch	MAX. UNIT THRUST		
		P =1000 Lb.	P =1500 lb.	P =2000 lb.
1	5	118.4	186.7	275.3
	7.5	69.5	118.8	169.0
	10	36.2	85.0	119.0
2	5	115.7	207.8	339.8
	7.5	71.8	125.7	192.8
	10	44.5	76.3	111.8
	12.5	41.0	64.2	92.6

TABLE 2.3 EXPERIMENTAL MAX. THRUST

CONDUIT NO	SOIL COVER H – Inch	BENDING MOMENT Ma – lb.in.		
		P =1000 lb.	P =1500 lb.	P =2000 lb.
1	5	52.4	78.9	106.5
	7.5	30.7	52.2	79.4
	10	21.7	35.8	54.5
2	5	23.1	38.1	60.5
	7.5	16.9	30.2	46.5
	10	9.2	16.2	25.6
	12.5	8.4	14.1	22.3

TABLE 2.4 BENDING MOMENT AT A (CROWN LEVEL) M_a

CONDUIT NO	SOIL COVER H – Inch .	BENDING MOMENT Mb – lb.in.		
		P =1000 lb.	P =1500 lb.	P =2000 lb.
1	5	-19.9	-31.0	-40.9
	7.5	-15.8	-24.0	-32.9
	10	-11.5	-20.1	-26.6
2	5	-12.60	-21.3	-31.2
	7.5	- 9.1	-15.5	-23.2
	10	- 5.3	- 9.6	-14.7
	12.5	- 4.9	- 8.1	-12.1

TABLE 2.5 BENDING MOMENT AT B (ie AT $\theta = 36^\circ$ LOCATION) M_b

CONDUIT #	H Inch	ALPHA Deg.	BETA Deg.	P Lb.	H* Inch	2wc Inch	2bc Inch	AREA In^2	p psi.	T_{max}
1	5	27	28	1000	4.70	12.0	9.80	118.0	8.47	127.0
				1500	4.55	11.8	9.64	114.3	13.10	196.8
				2000	4.40	11.7	9.50	111.2	17.98	269.7
	7.5	27	34	1000	7.20	16.7	12.34	206.2	4.84	72.7
				1500	7.05	16.5	12.20	201.4	7.45	111.7
				2000	6.90	16.3	12.03	196.2	10.19	152.9
	10	23	42	1000	9.70	24.5	13.30	326.0	3.06	45.9
				1500	9.55	24.2	13.11	317.3	4.72	70.8
				2000	9.40	24.0	13.00	312.0	6.40	96.0
2	5	24	24	1000	4.70	11.2	9.20	103.1	9.70	145.5
				1500	4.55	11.1	9.10	101.0	14.80	222.0
				2000	4.40	11.0	9.00	99.0	20.20	303.0
	7.5	20	31	1000	7.20	15.7	10.30	161.7	6.18	92.7
				1500	7.05	15.5	10.13	157.0	9.55	143.2
				2000	6.90	15.3	10.03	153.5	13.00	195.0
	10	16	55	1000	9.70	34.7	10.60	368.0	2.71	40.7
				1500	9.55	34.3	10.50	360.2	4.16	62.4
				2000	9.40	33.85	10.40	352.0	5.68	85.2
	12.5	13	58	1000	12.20	46.0	10.64	489.0	2.04	30.6
				1500	12.05	45.6	10.60	484.0	3.10	46.5
				2000	11.90	45.1	10.50	474.0	4.22	63.30

TABLE 3.1 MAXIMUM THRUST CALCULATIONS BY PROPOSED THRUST EQUATION

NO.	SOIL COVER H - Inch	LOAD	EXPERIMENTAL	ORIGINAL IOWA FORMULA	DELTA 2	DELTA 3
1	5	1000	90	82	188	143
		1500	152	123	283	215
		2000	248	164	377	286
2	7.5	1000	65	63	114	90
		1500	107	94	171	135
		2000	170	126	228	180
3	10	1000	40	51	80	65
		1500	60	76	120	98
		2000	98	102	160	131
4	12.5	1000	28	43	61	51
		1500	42	64	92	77
		2000	55	86	123	103

TABLE 3.2 COMPARISON OF DEFLECTIONS (E=2,000 psi) $\delta \times 10^{-3}$ IN.

SOIL COVER in.	LOAD P, in.	H' in.	$2w_c$ in.	$2b_c$ in.	AREA in^2	PRESSURE p - psi.	CONDUIT #1		CONDUIT # 2	
							T-1	T-2	T-1	T-2
5	1000	4.70	16.4	9.70	159.08	6.29	94.40	51.60	97.50	51.60
	1500	4.55	16.1	9.55	153.76	9.76	146.40	78.53	151.30	78.53
	2000	4.40	15.8	9.40	148.52	13.47	202.00	106.40	208.80	106.40
7.5	1000	7.20	21.4	12.20	261.08	3.83	57.50	41.00	59.40	41.00
	1500	7.05	21.1	12.05	254.26	5.90	88.50	62.24	91.50	62.24
	2000	6.90	20.8	11.90	247.52	8.08	121.20	84.03	125.20	84.03
10	1000	9.70	26.4	14.70	388.08	2.58	38.70	34.01	40.00	34.01
	1500	9.55	26.1	14.55	379.76	4.00	60.00	51.55	62.00	51.55
	2000	9.40	25.8	14.40	371.52	5.40	81.00	69.44	83.70	69.44
12.5	1000	12.20	31.4	17.20	540.08	1.85	27.80	27.77	28.70	28.70
	1500	12.05	31.1	17.05	530.26	2.83	42.45	42.43	43.87	43.87
	2000	11.90	30.8	16.90	520.52	3.84	57.63	57.63	59.50	59.17

$T_{-1} = p \cdot R$, $T_{-2} = P'/2 = (P/2b_c) \times 1/2$

TABLE 4.1 UNIT THRUST CALCULATIONS BY OHBDC PROVISIONS

SOIL COVER H-Inch	LOAD P lbs	H* Inch	$2w_c$ Inch	2bc Inch	AREA in^2	PRESSURE p psi	T = p.R. lb/in. CONDUIT #1	CONDUIT #2
5	1000	4.70	15.23	13.23	201.4	4.97	74.6	77.0
	1500	4.55	14.96	12.96	193.9	7.74	116.1	120.0
	2000	4.40	14.70	12.70	186.7	10.71	160.7	166.0
7.5	1000	7.20	19.60	17.60	345.0	2.90	43.5	45.0
	1500	7.05	19.34	17.34	335.3	4.47	67.1	69.3
	2000	6.90	19.08	17.08	325.9	6.14	92.0	95.1
10	1000	9.70	23.98	21.98	527.1	1.90	28.5	29.4
	1500	9.55	23.71	21.71	514.7	2.90	43.7	45.2
	2000	9.40	23.45	21.45	503.0	3.98	59.6	61.6
12.5	1000	12.20	28.35	26.35	747.0	1.34	20.1	20.8
	1500	12.05	28.09	26.09	732.9	2.05	30.7	31.7
	2000	11.90	27.83	25.83	719.0	2.78	41.7	43.1

TABLE 4.2 MAXIMUM UNIT THRUST BY AASHTO

CONDUIT NO	SOIL COVER H-Inch	LOAD P Lbs.	MAXIMUM UNIT THRUST (lb/inch) BY				
			EXPERIMENT	AASHTO	OHBDC		PROPOSED
					T-1	T-2	
1	5	1000	118.4	74.6	94.4	51.6	127.0
		1500	186.7	116.1	146.4	78.5	196.8
		2000	275.3	160.7	202.0	106.4	269.7
	7.5	1000	69.5	43.5	57.5	41.0	72.7
		1500	118.8	67.1	88.5	62.2	111.7
		2000	169.0	92.1	121.2	84.0	152.9
	10	1000	36.2	28.5	38.7	34.0	45.9
		1500	85.0	43.7	60.0	51.6	70.8
		2000	119.0	59.6	81.0	69.4	96.0
2	5	1000	115.7	77.0	97.5	51.6	145.5
		1500	207.8	120.0	151.3	78.5	222.0
		2000	339.8	166.0	208.8	106.4	303.0
	7.5	1000	71.8	45.0	59.4	41.0	92.7
		1500	125.7	69.3	91.5	62.2	143.2
		2000	192.8	95.1	125.2	84.0	195.0
	10	1000	45.0	29.4	40.0	34.0	40.7
		1500	76.3	45.2	62.0	51.6	62.4
		2000	111.8	61.6	83.7	69.4	85.2
	12.5	1000	41.0	28.7	28.7	28.7	30.6
		1500	63.2	31.7	43.9	43.9	46.5
		2000	92.6	43.1	59.5	59.2	63.3

TABLE 4.3 COMPARISON OF MAXIMUM UNIT THRUST OBTAINED BY VARIOUS METHODS

CONDUIT NO.	SOIL H-Inch.	LOAD P Lb.	PRESSURE (psi) AT CROWN LEVEL BY			
			EXPERIMENT	AASHTO	OHBDC	PROPOSED
1	5	1000	7.20	4.97	6.30	8.47
		1500	13.20	7.74	9.80	13.12
		2000	21.60	10.71	13.50	18.00
	7.5	1000	4.20	2.90	3.83	4.85
		1500	7.20	4.47	5.90	7.45
		2000	11.60	6.14	8.10	10.20
	10	1000	2.80	1.90	2.60	3.06
		1500	5.15	2.91	4.00	4.72
		2000	6.75	4.00	5.40	6.40
2	5	1000	10.40	4.97	6.29	9.70
		1500	15.00	7.74	9.76	14.80
		2000	20.40	10.71	13.47	20.20
	7.5	1000	6.20	2.90	3.83	6.18
		1500	9.20	4.47	5.90	9.55
		2000	13.20	6.14	8.10	13.00
	10	1000	2.45	1.90	2.60	2.71
		1500	4.20	2.91	4.00	4.16
		2000	6.25	4.00	5.40	5.68
	12.5	1000	2.04	1.34	1.90	2.04
		1500	3.13	2.05	2.83	3.10
		2000	4.54	2.78	3.84	4.22

TABLE 4.4 COMPARISON OF LIVE LOAD PRESSURE AT THE CROWN LEVEL OF CONDUIT - BY VARIOUS METHODS.

FIGURES

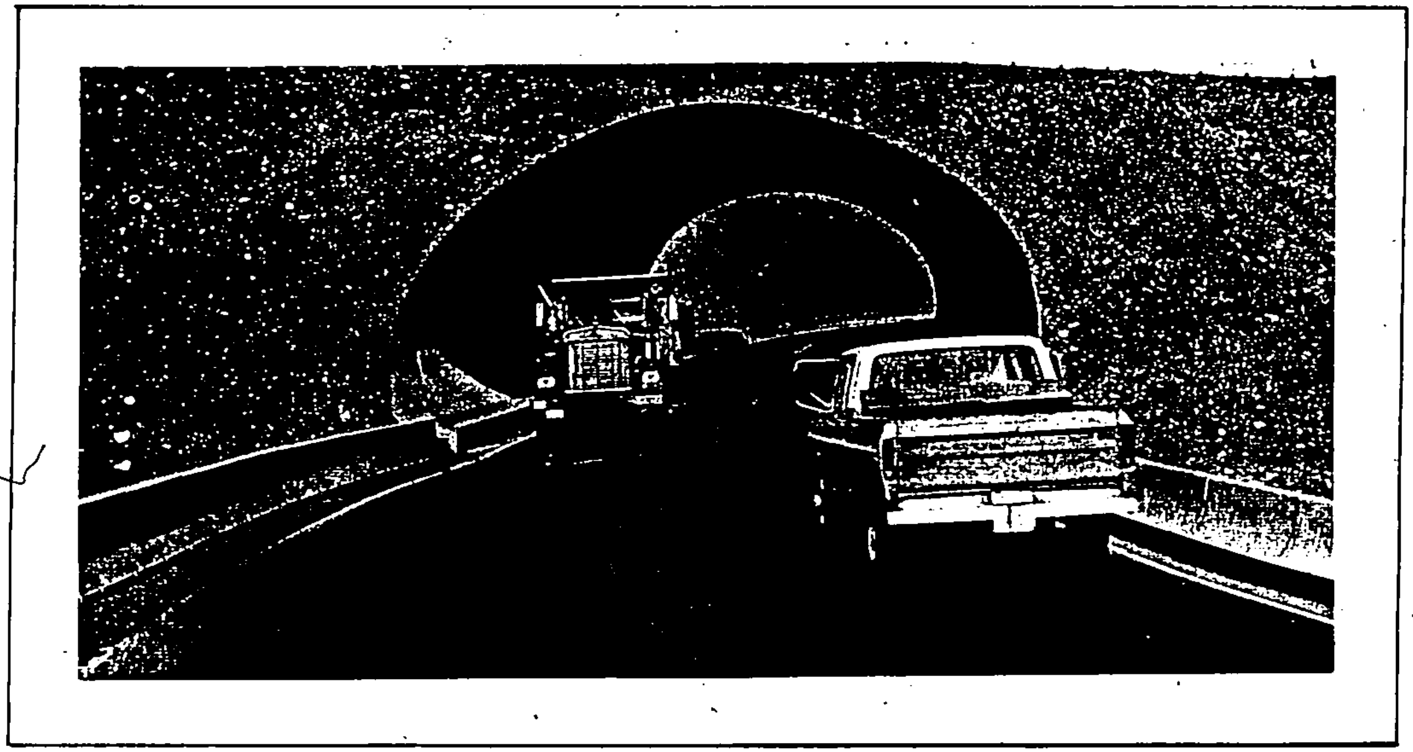


Fig. 1.1 TYPICAL SOIL-STEEL STRUCTURE

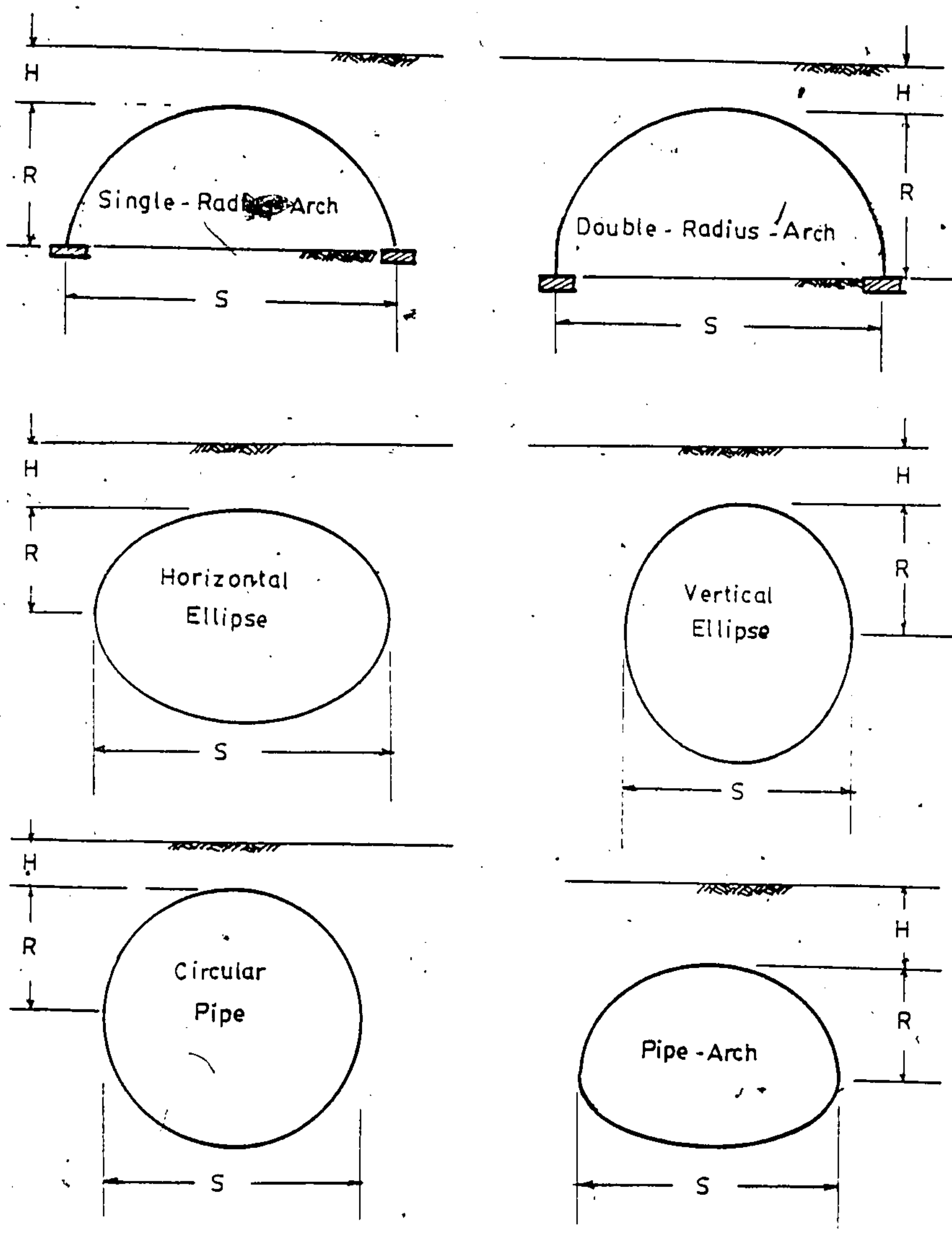


Fig. 1.2 TYPES OF METAL CULVERT STRUCTURES

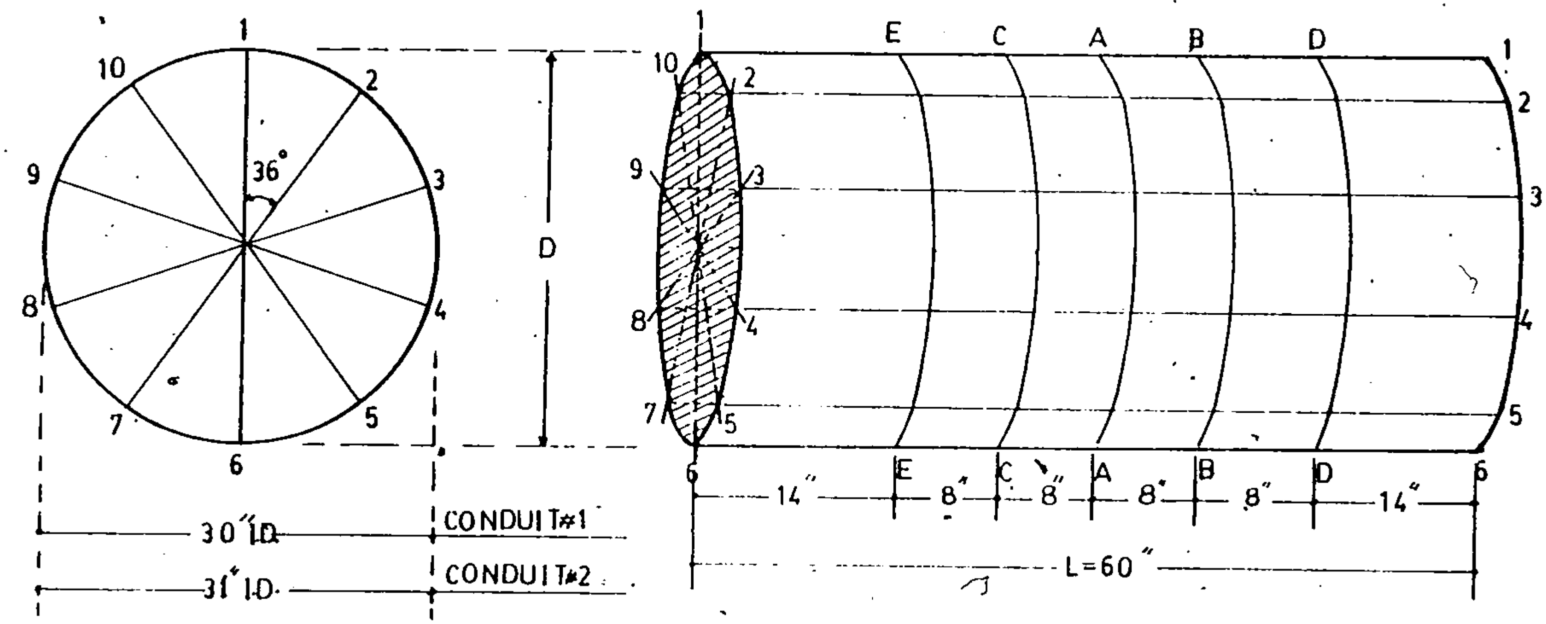
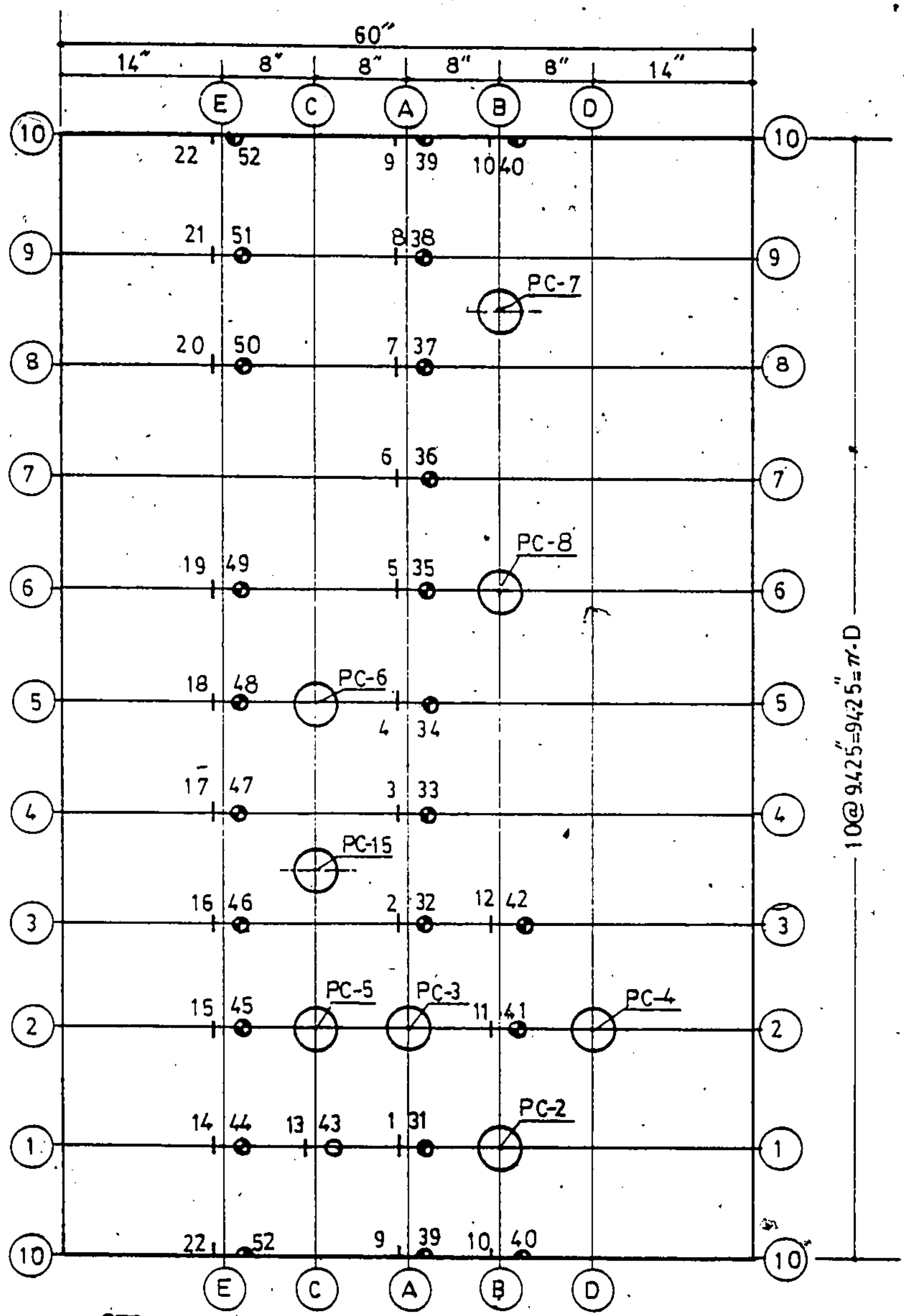


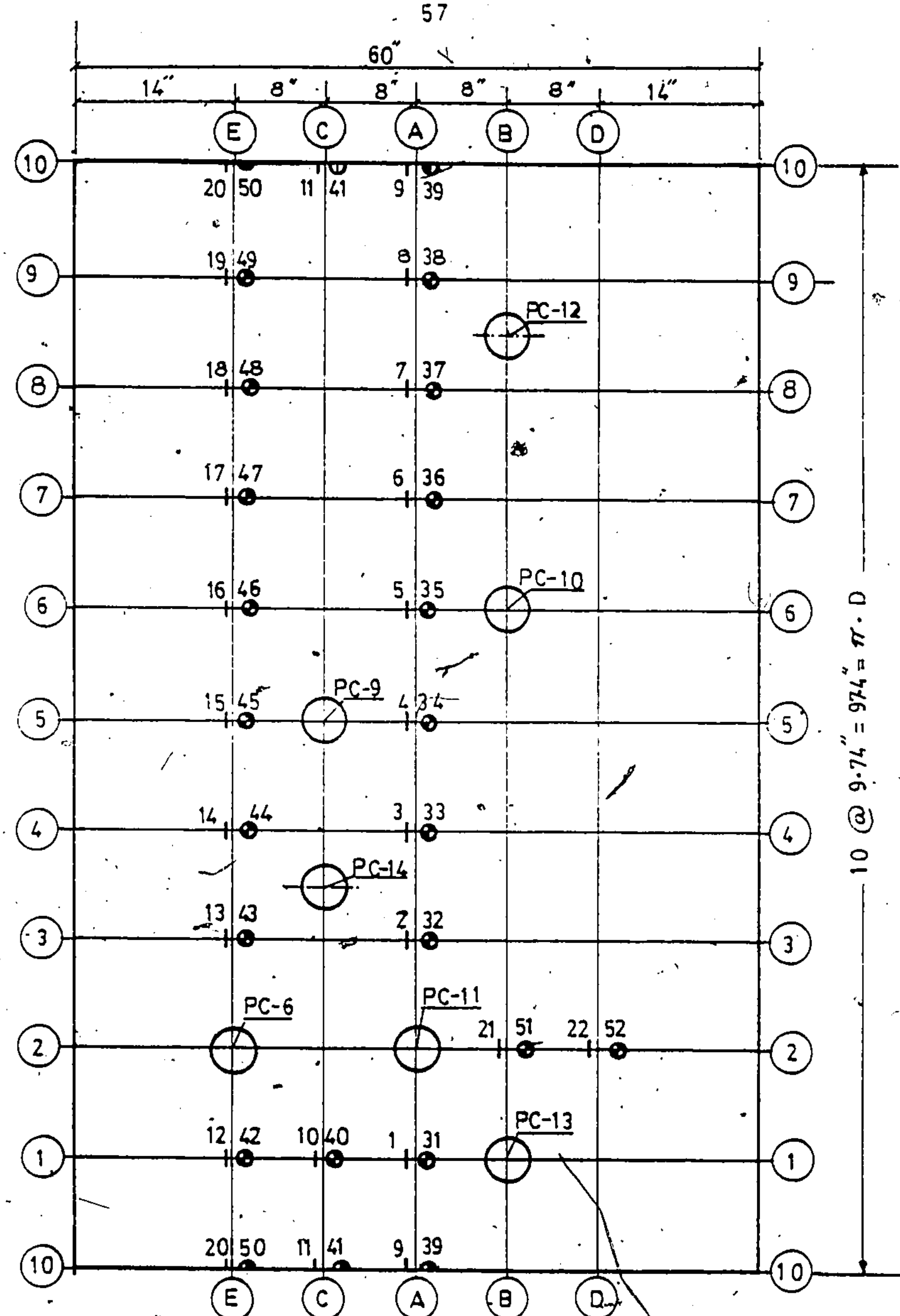
FIG.2.1. DETAILS OF CONDUIT MODELS.



— STRAIN GAGES INSTALLED ON ~~OUTER~~ SURFACE

● STRAIN GAGES INSTALLED ON INNER SURFACE

FIG.2.2. DEVELOPED PLAN OF CONDUIT #1 WITH INSTRUMENTATION DETAILS



STRAIN GAGES INSTALLED ON OUTER SURFACE

STRAIN GAGES INSTALLED ON INNER SURFACE

FIG.2.3 DEVELOPED PLAN OF CONDUIT #2 WITH INSTRUMENTATION DETAILS

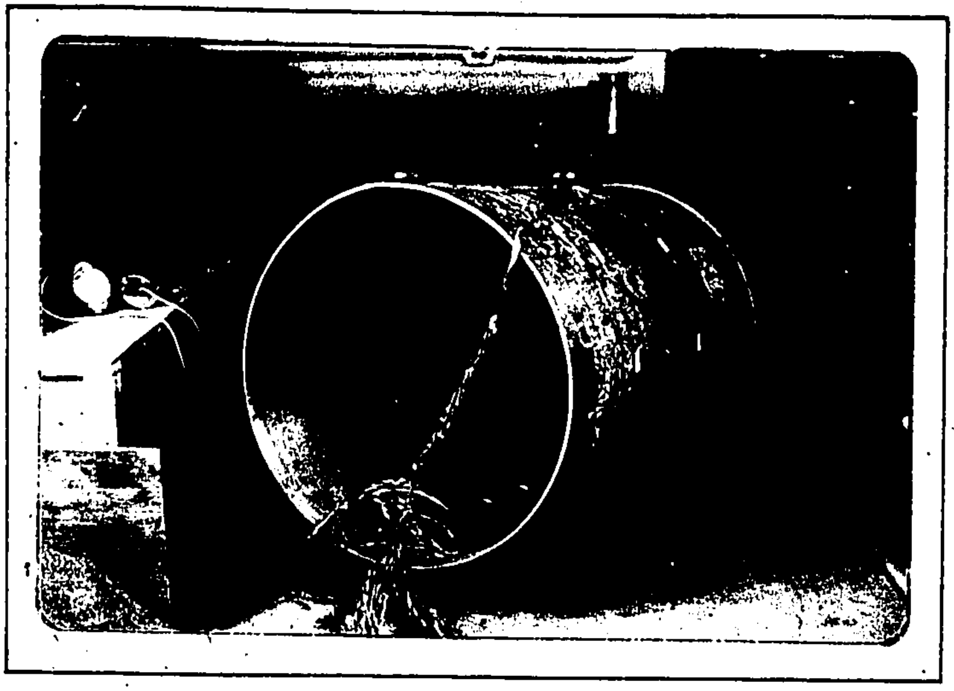


FIG. 2.4. TEST MODEL-CONDUIT #1

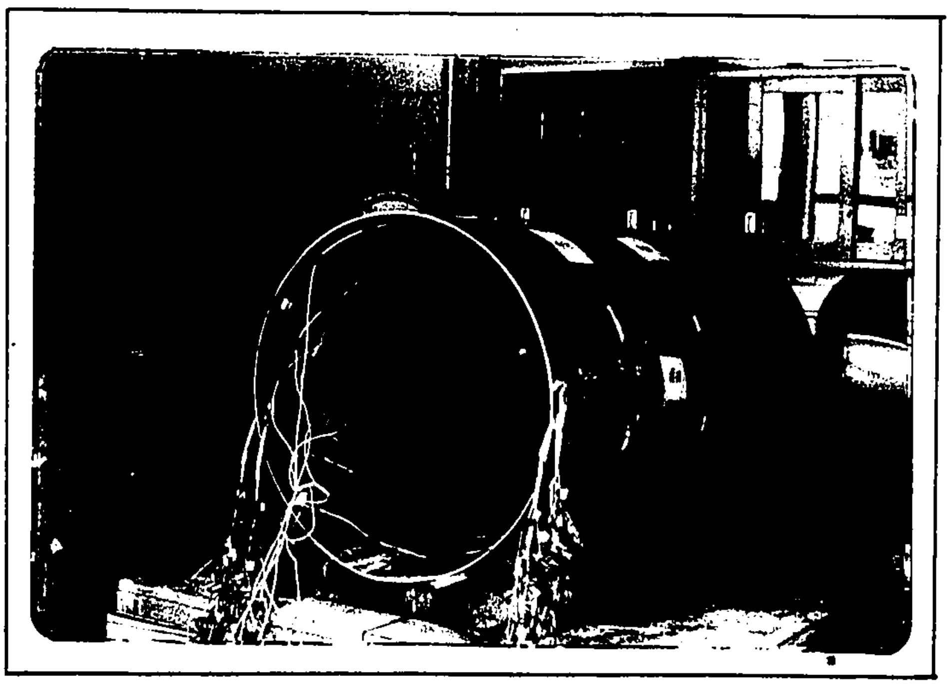


FIG. 2.5 TEST MODEL-CONDUIT #2

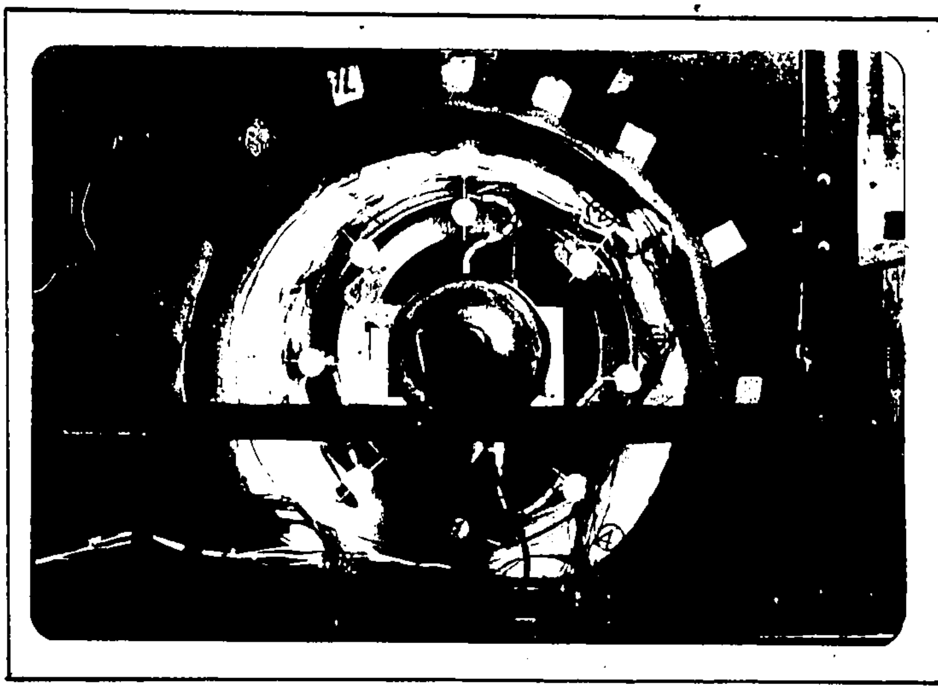


FIG.2.6 DEFLECTION GAGE INSTALLATION FRAME.

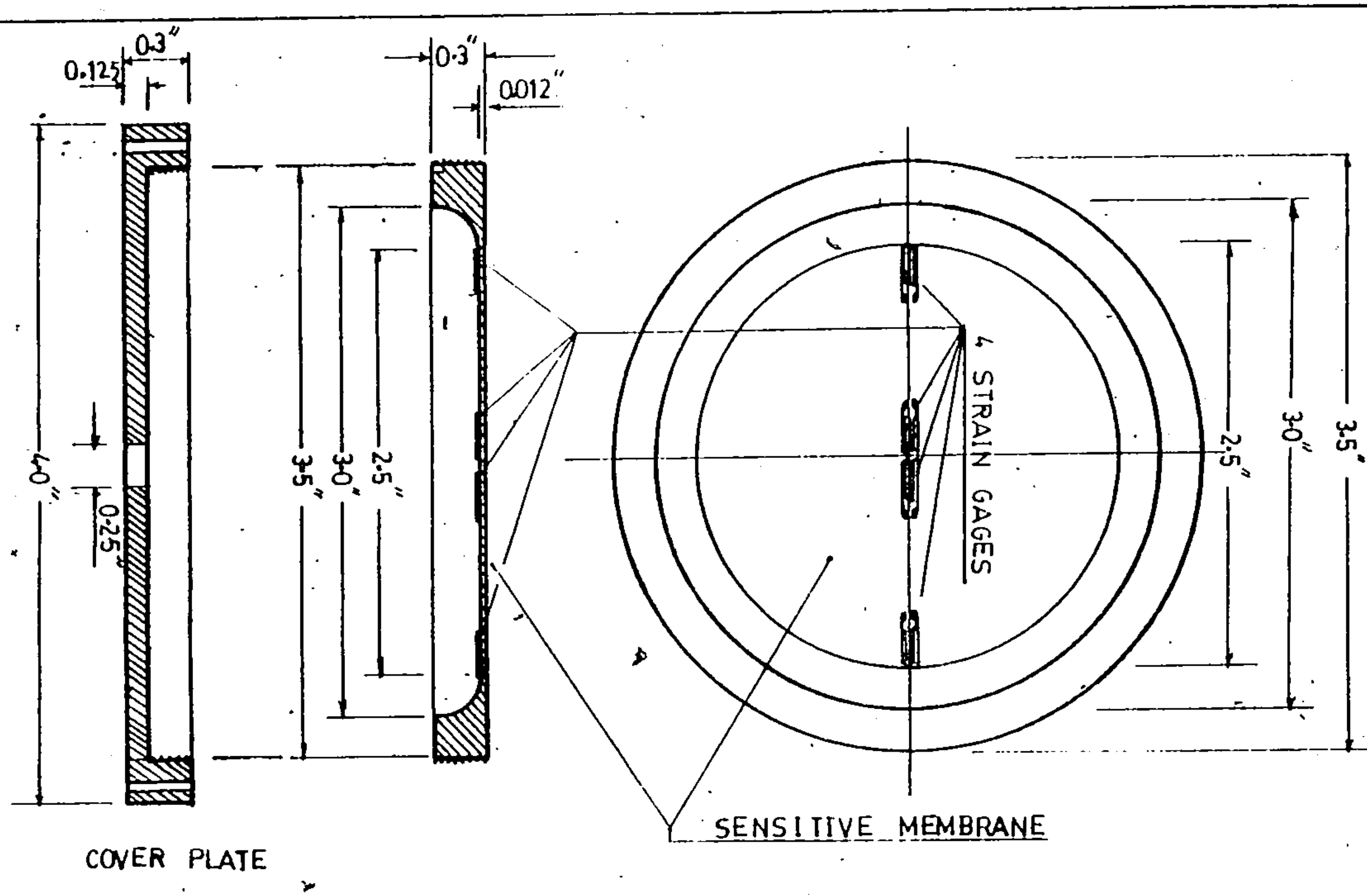


FIG. 2.7 CONSTRUCTION DETAILS OF PRESSURE CELL

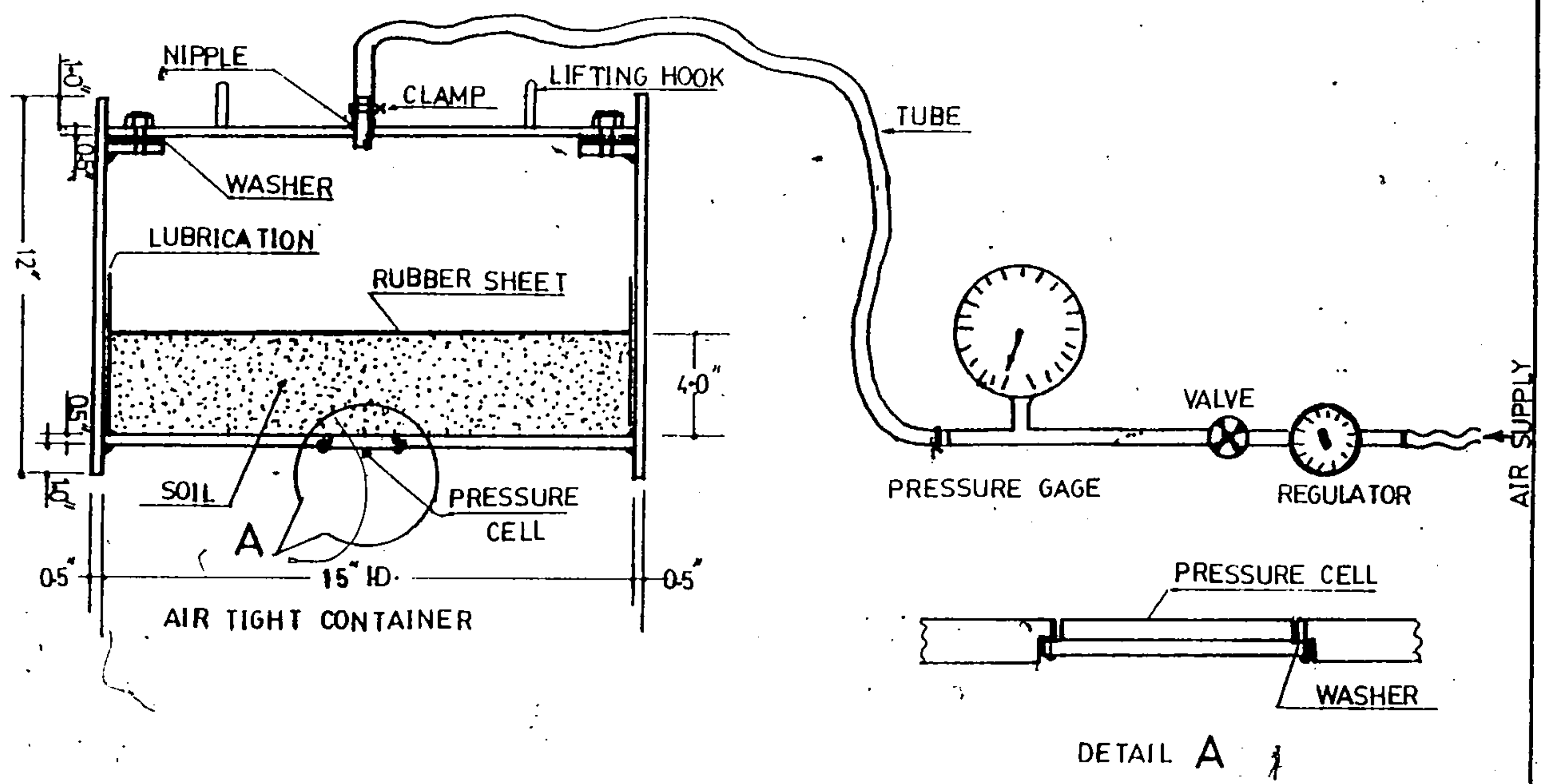


FIG.2.8 PRESSURE CELL CALIBRATION DEVICE

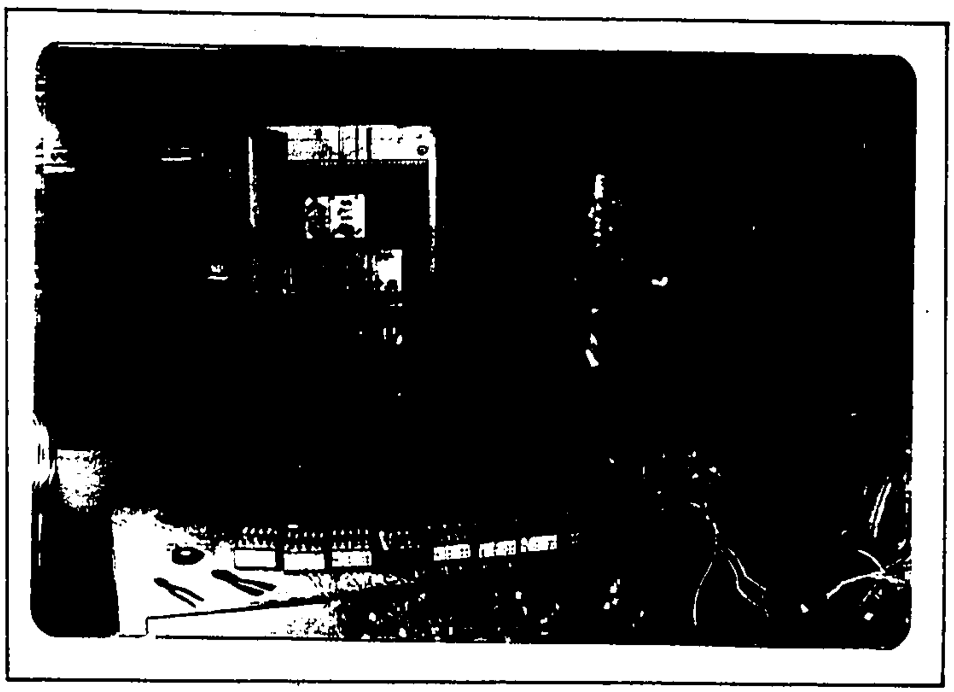


FIG. 2.9 MULTICHANNEL DIGITAL STRAIN INDICATOR

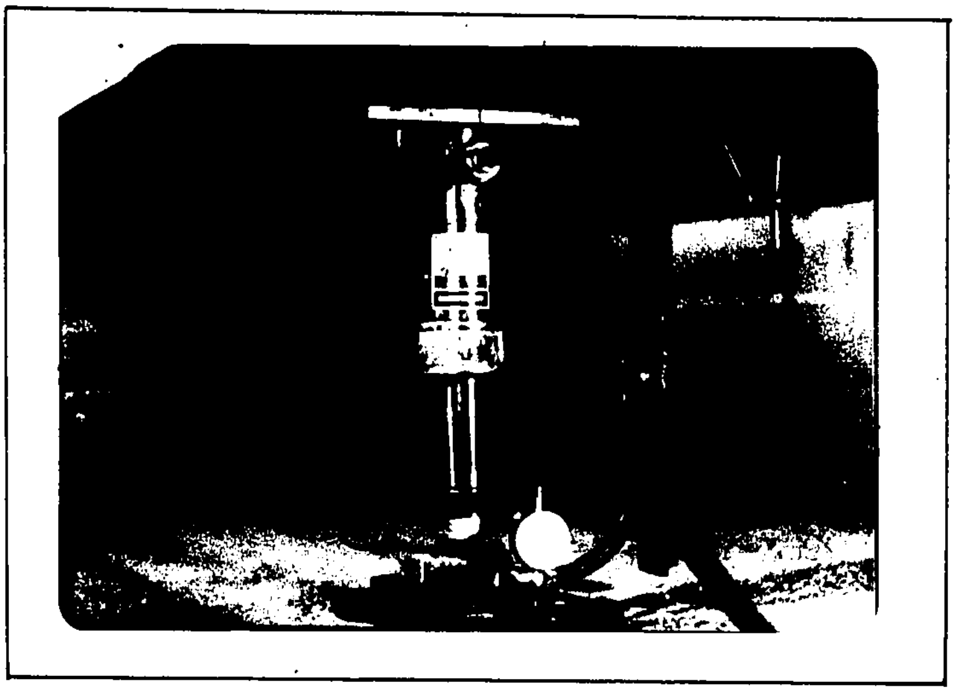


FIG. 2.10 LOAD CELL

FIG. 2.11 BOX SHAPED TEST CELL

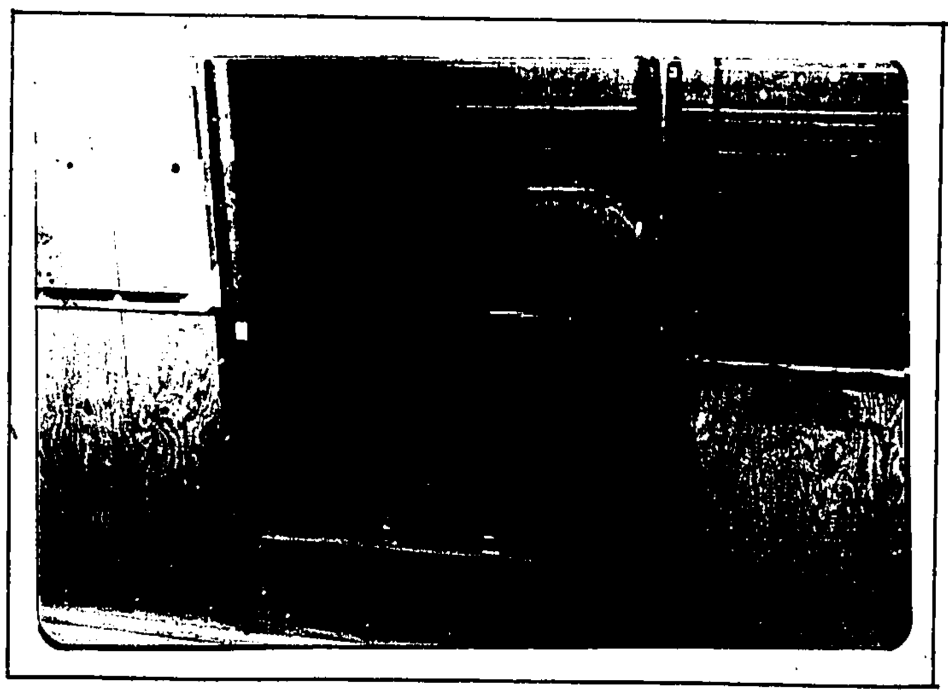


FIG. 2.12 - 12 INCH DEEP SOIL BED

(a)

(b)

FIG. 2.13 CONDUIT IN THE CONSTRUCTION STAGE

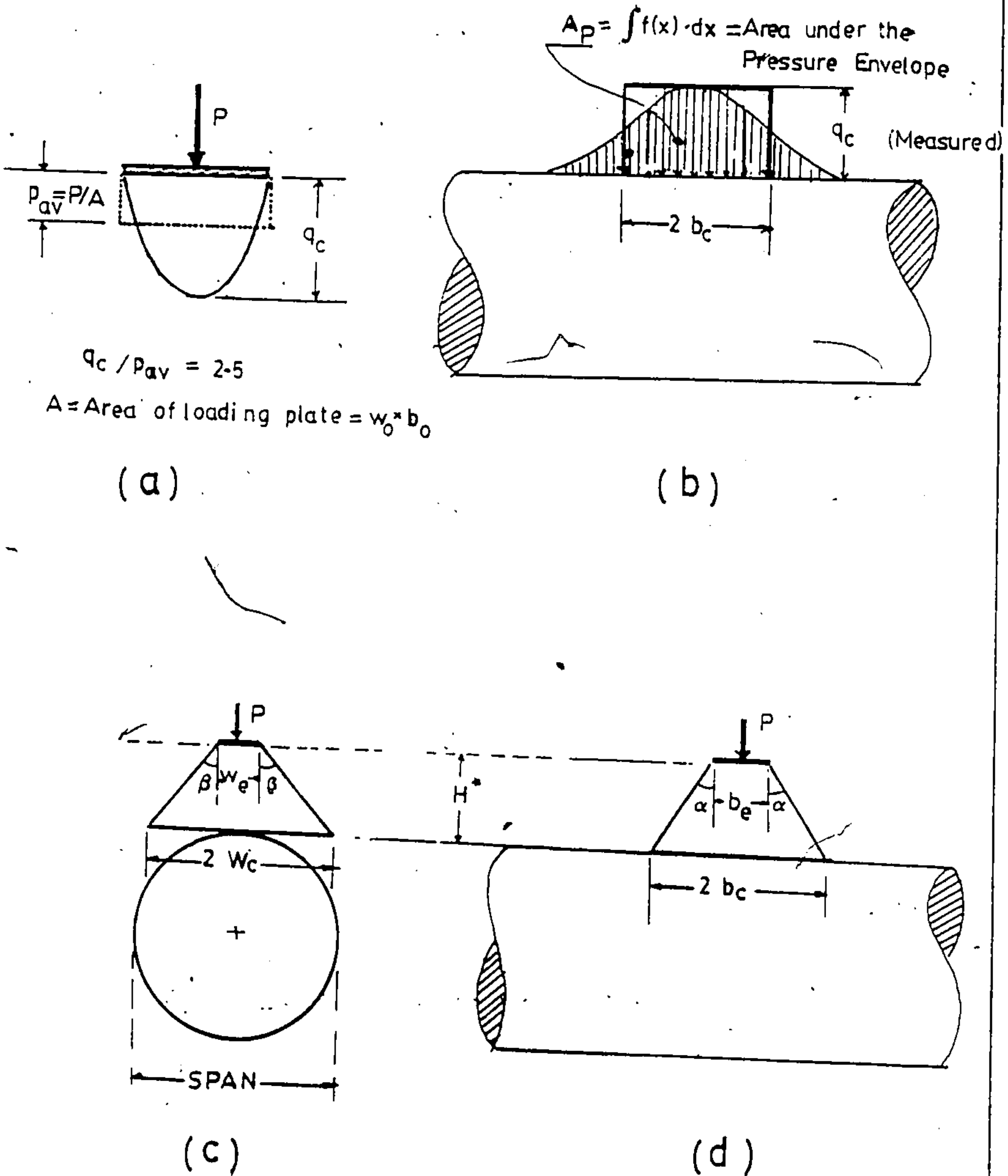


Fig - 2.14 LOAD DISTRIBUTION ANGLES

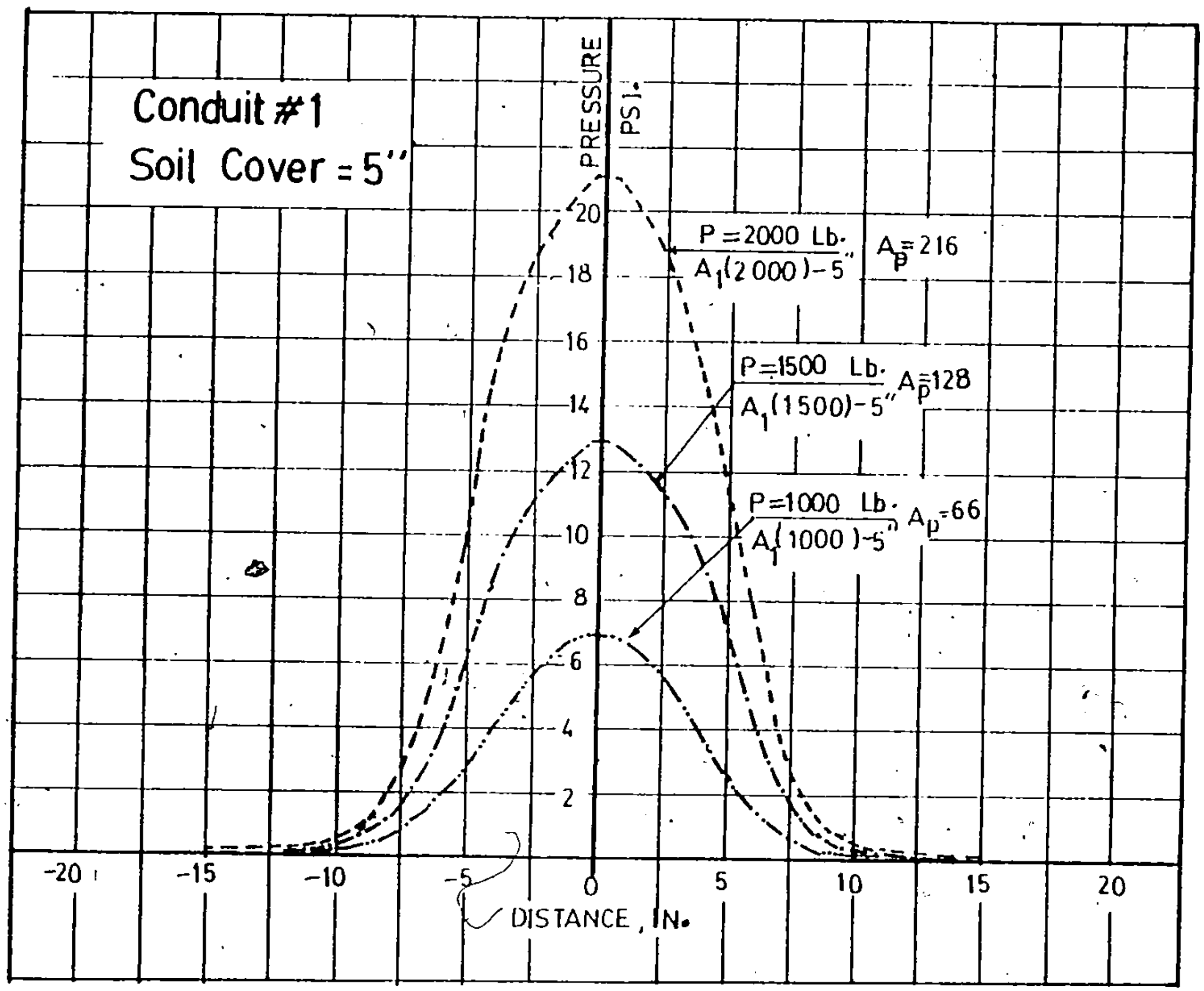


FIG. 2.15 PRESSURE ENVELOPE AT CROWN LEVEL - FOR A_1(L) -5", CONDUIT #1

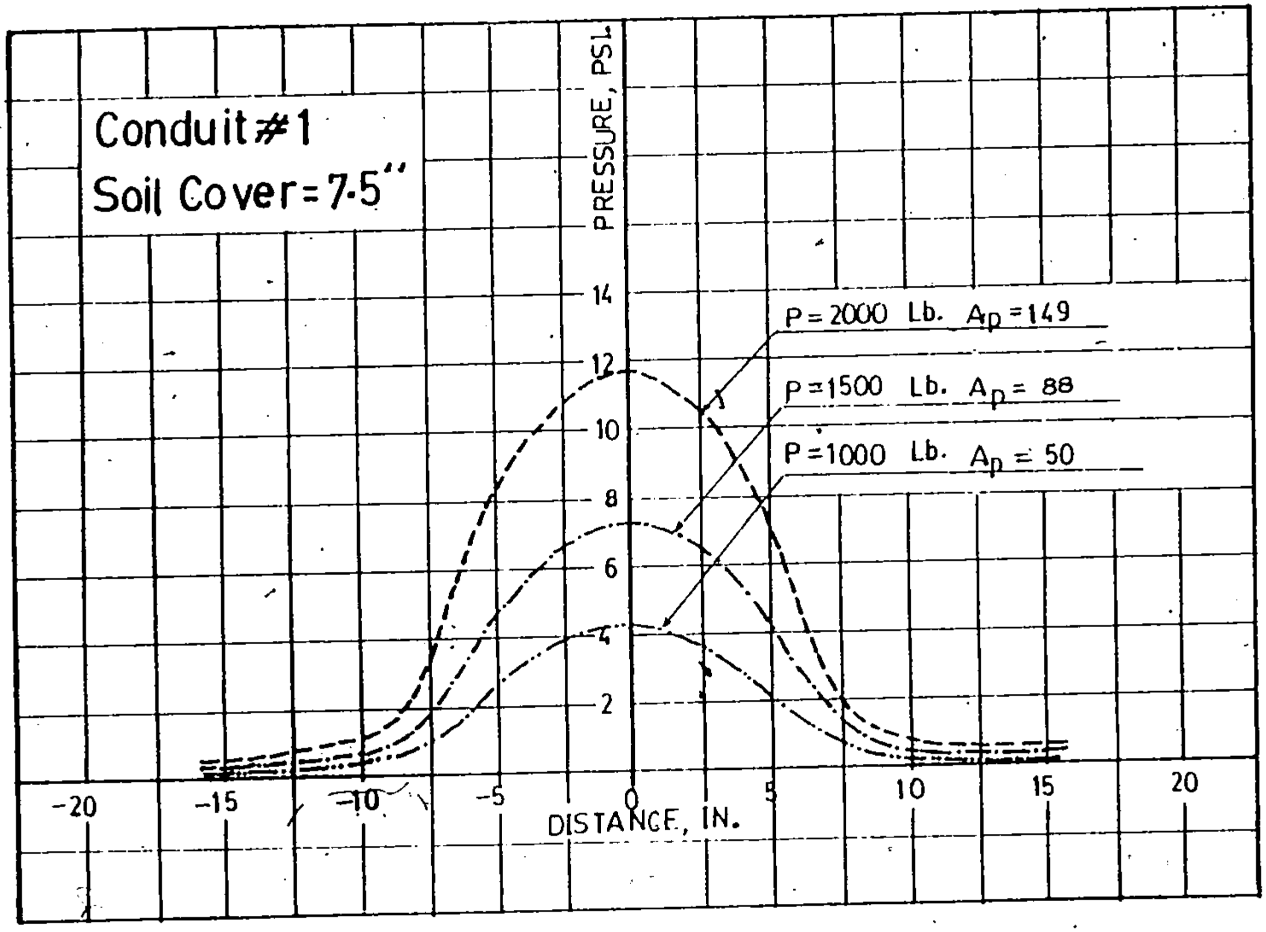


FIG.2.16 PRESSURE ENVELOPE AT CROWN LEVEL - FOR A_1' (L) - 7.5", CONDUIT #1

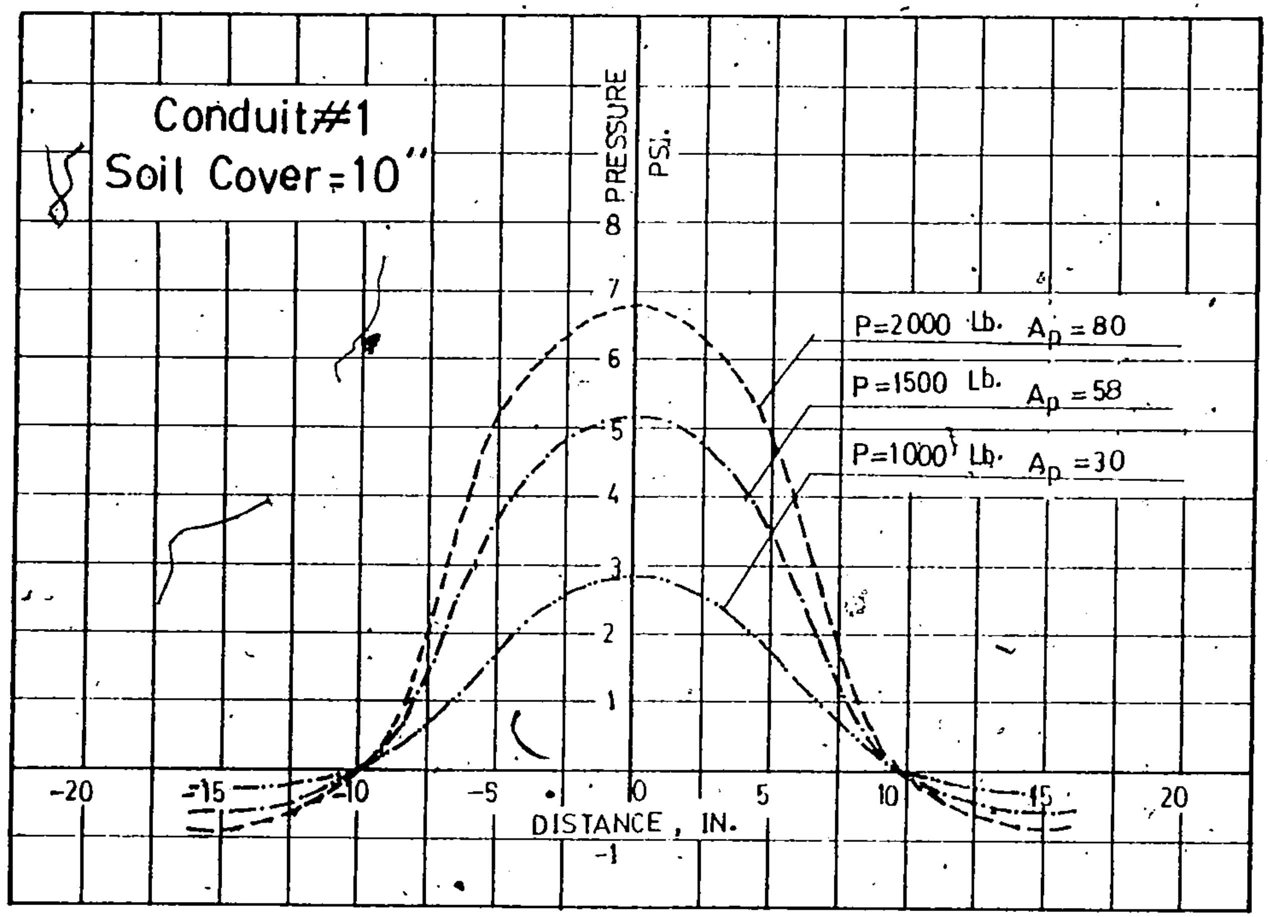


FIG. 2.17 PRESSURE ENVELOPE AT CROWN LEVEL - FOR A_1 (L) - 10", CONDUIT #1

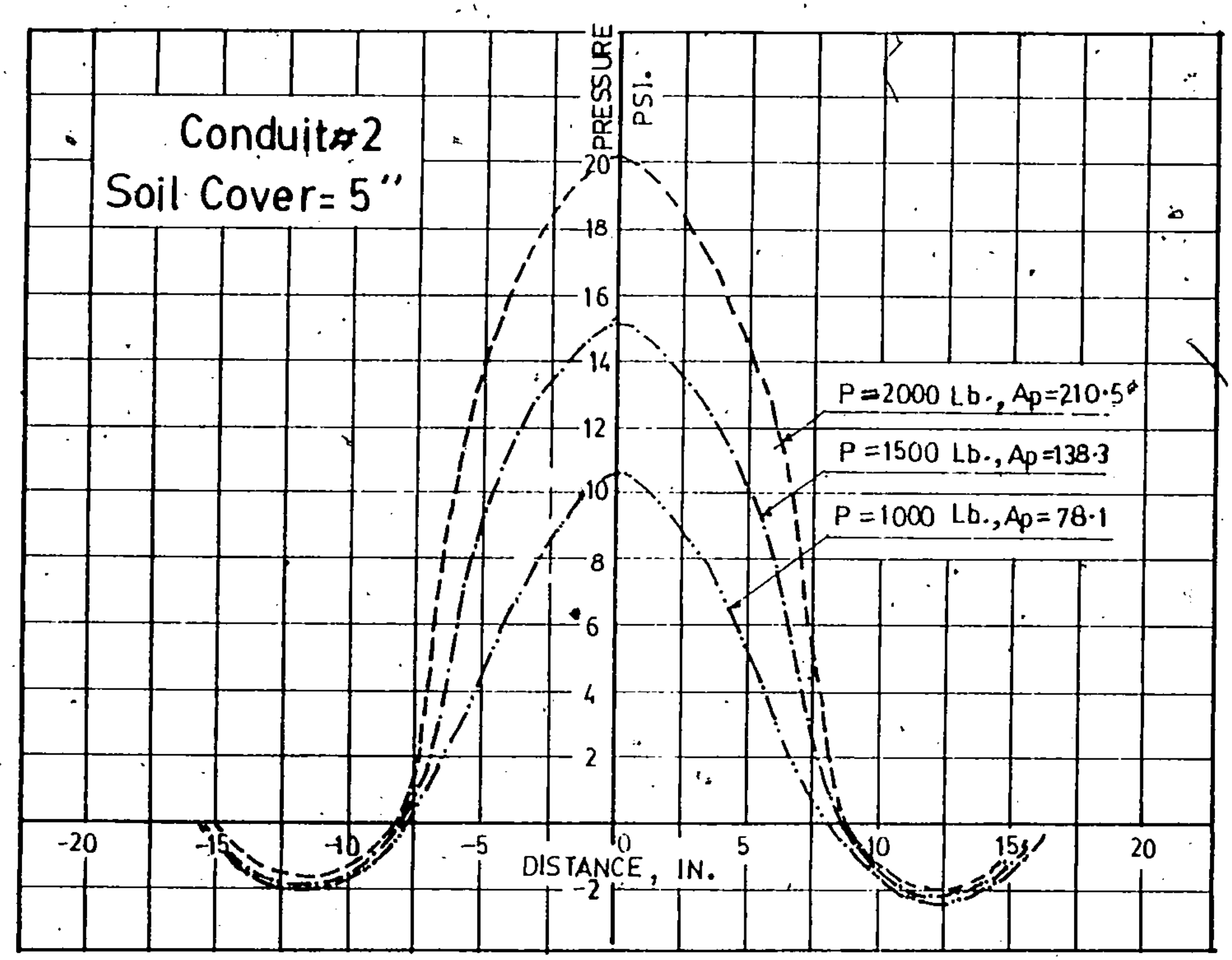


FIG. 2.18 PRESSURE ENVELOPE AT CROWN LEVEL - FOR A_1 (L) - 5", CONDUIT #2

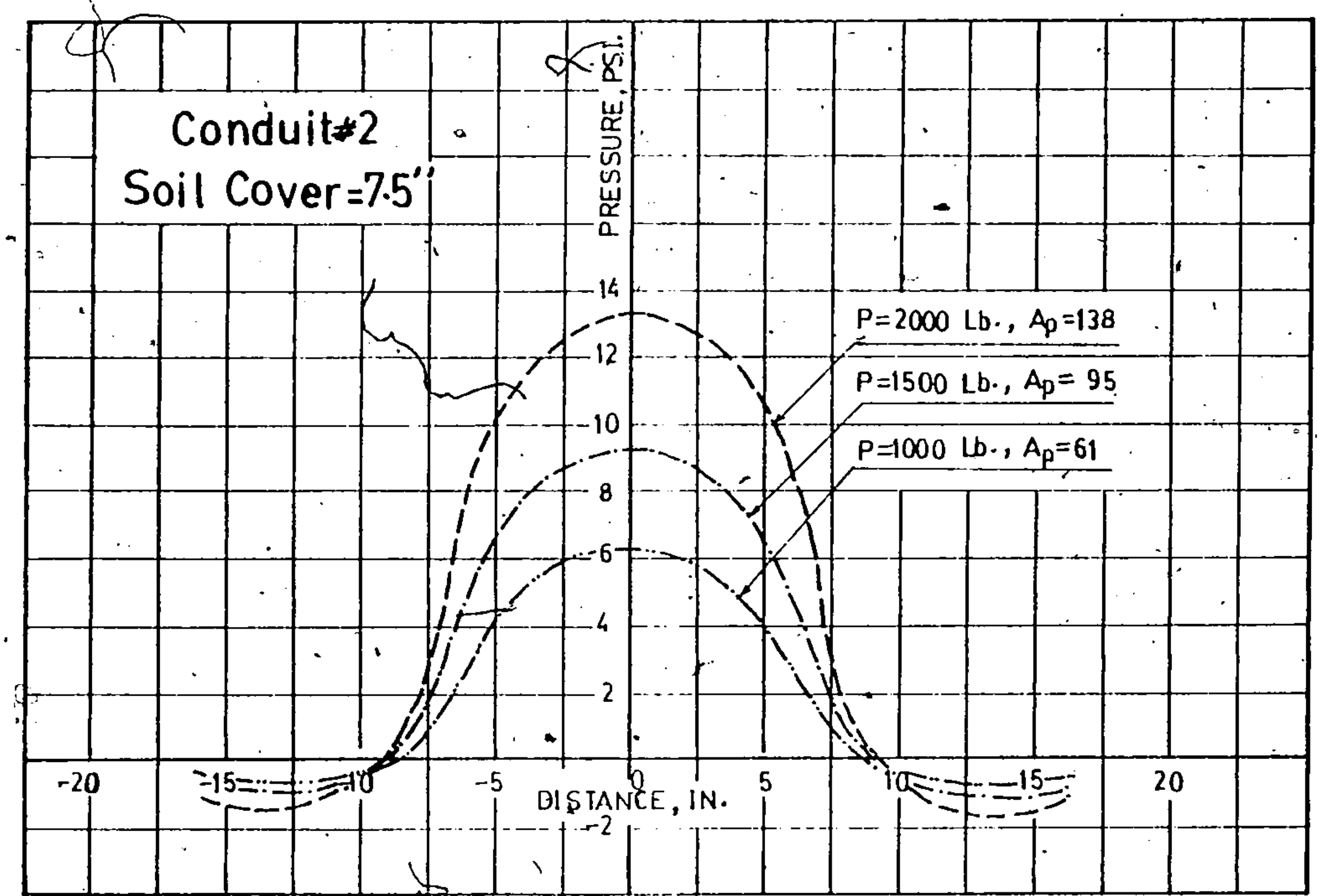


FIG. 2.19 PRESSURE ENVELOPE AT CROWN LEVEL - FOR A_1 (L) - 7.5" CONDUIT #2

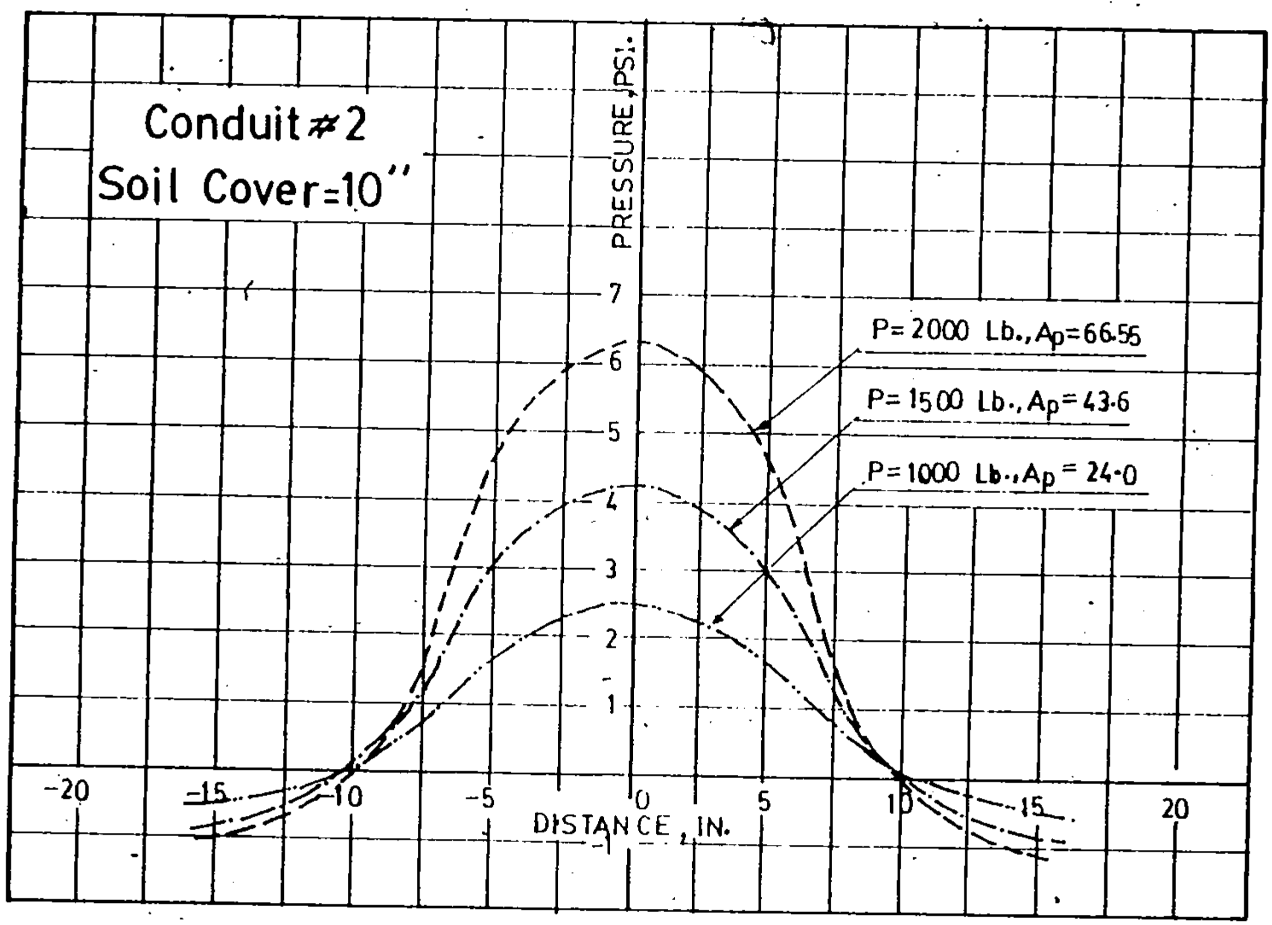


FIG. 2.20 PRESSURE ENVELOPE AT CROWN LEVEL - FOR A_1 (L) - 10" , CONDUIT #2

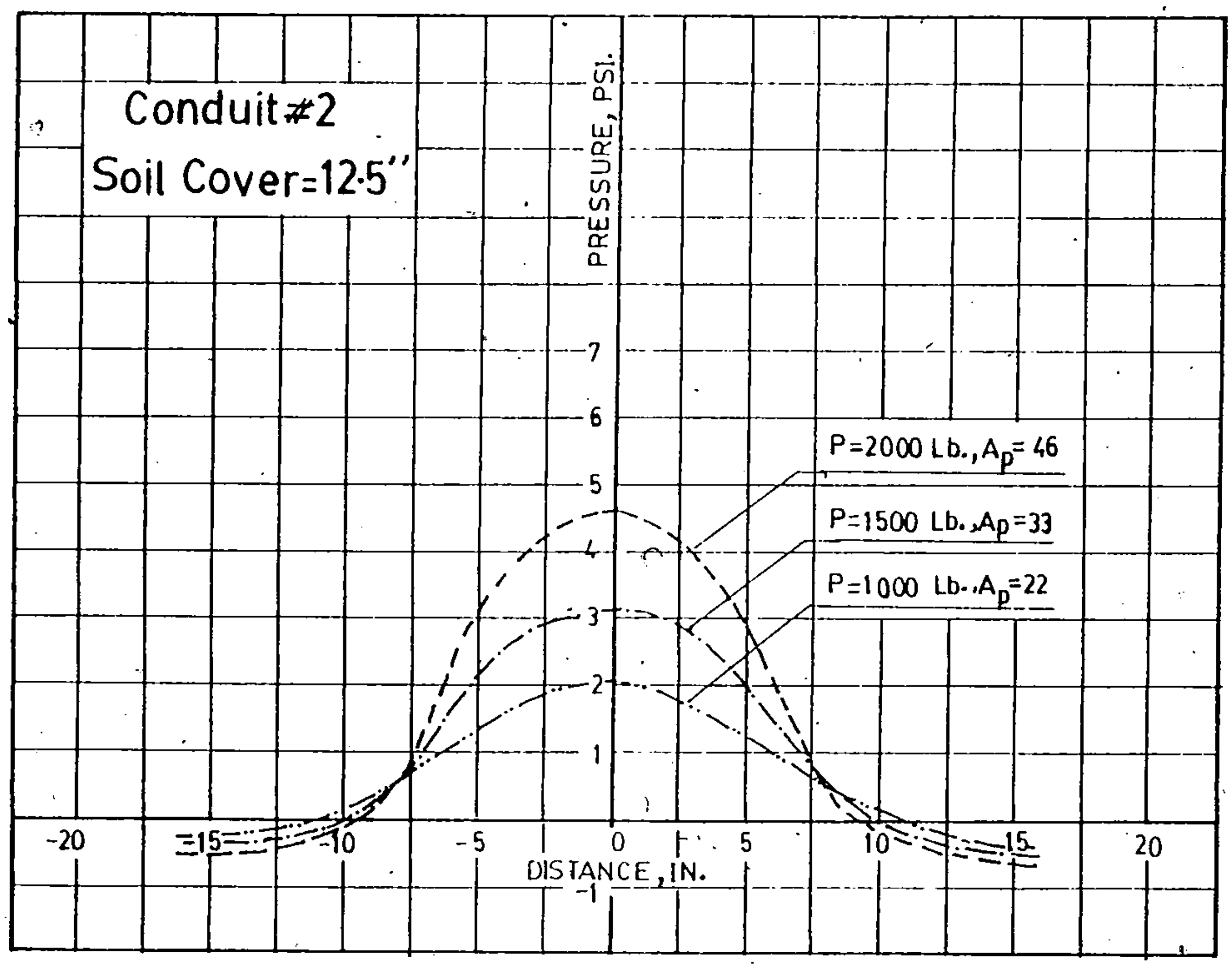
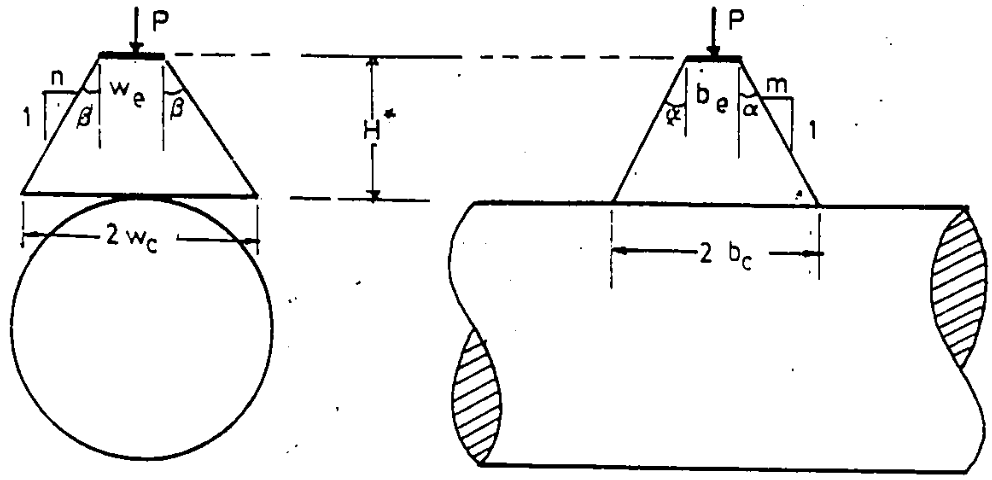


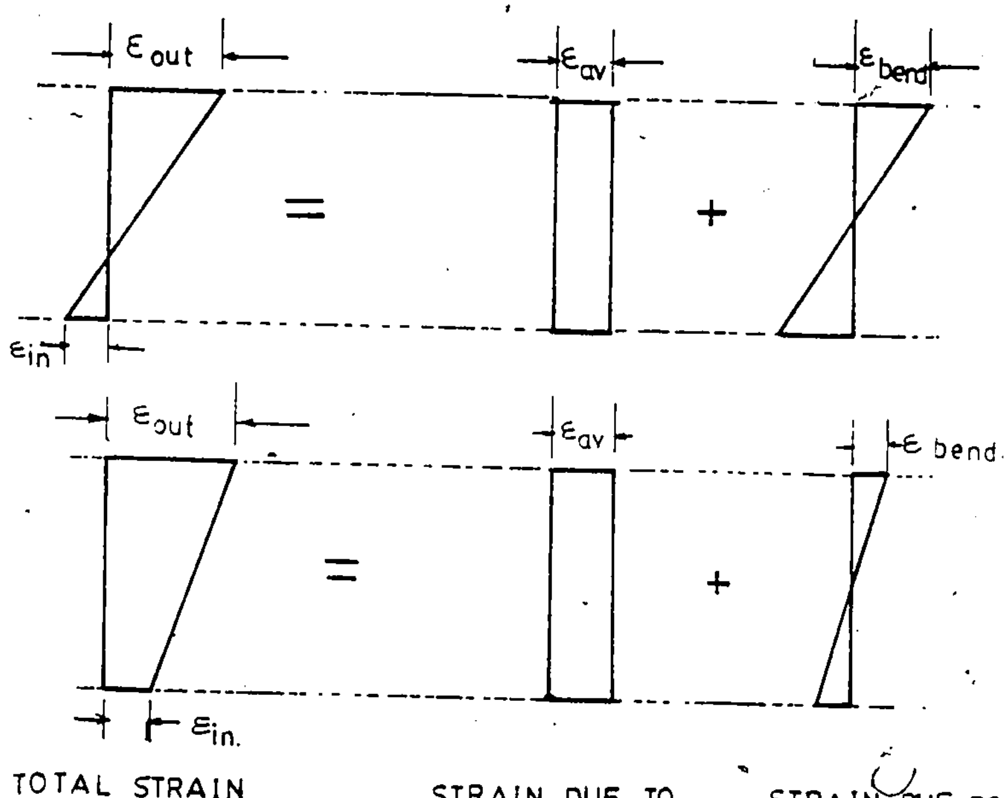
FIG. 2.21 PRESSURE ENVELOPE AT CROWN LEVEL - FOR A_1 (L) - 12.5" , CONDUIT #2



NO.	DESCRIPTION	AASHTO	OHBD
1	m	0.875	0.5
2	n	0.875	1
3	$2\,b_c$	$b_e + 1.75\,H^*$	$b_e + H^*$
4	$2\,w_c$	$w_e + 1.75\,H^*$	$b_e + 2\,H^*$
5	α	$\tan^{-1}\left[\frac{2\,b_c - b_e}{2\,H^*}\right] = 41.2^\circ$	$\tan^{-1}\left[\frac{2\,b_c - b_e}{2\,H^*}\right] = 26.6^\circ$
6	β	$\tan^{-1}\left[\frac{2w_c - w_e}{2\,H^*}\right] = 41.2^\circ$	$\tan^{-1}\left[\frac{2w_c - w_e}{2\,H^*}\right] = 45^\circ$

$H^* = H - 0.30$ FOR $P = 1000$ Lb.
$= H - 0.45$ FOR $P = 1500$ Lb.
$= H - 0.60$ FOR $P = 2000$ Lb.

FIG.2.22 LOAD DISPERSION ANGLES SPECIFIED BY OHBDC AND AASHTO



$\varepsilon_{av} = [\varepsilon_{out} + \varepsilon_{in}] / 2$; $\varepsilon_{bend} = \varepsilon_{in} - \varepsilon_{av}$.

Fig. 2.23 MEASURED STRAIN SPLIT INTO BENDING AND AXIAL STRAIN

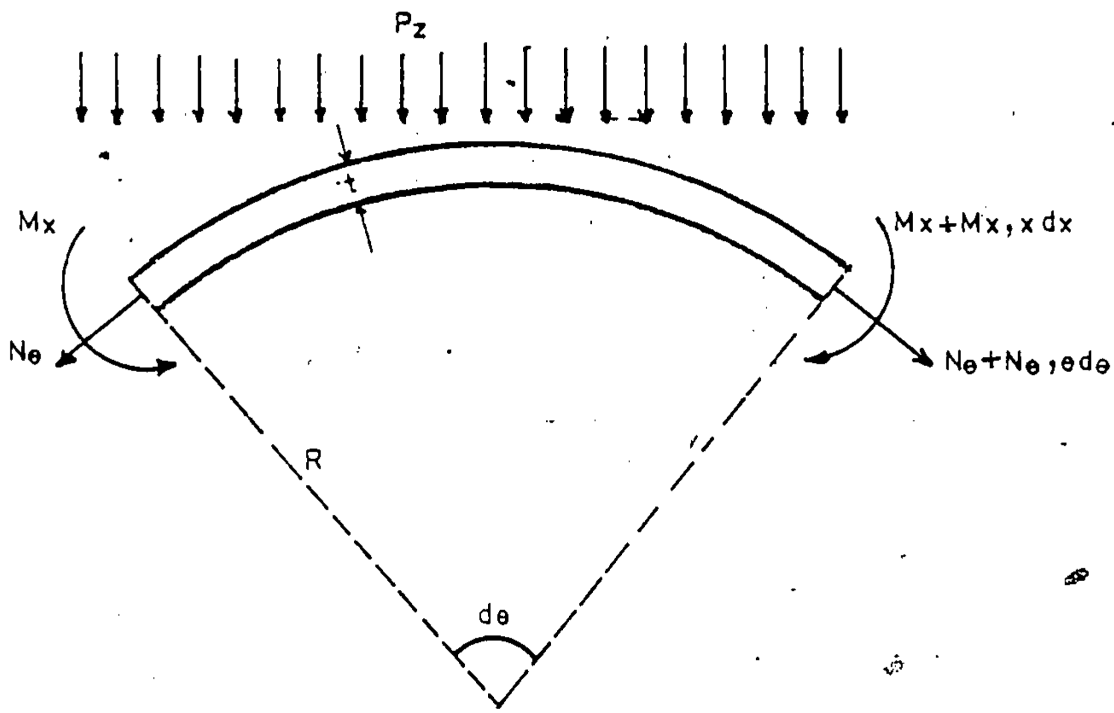
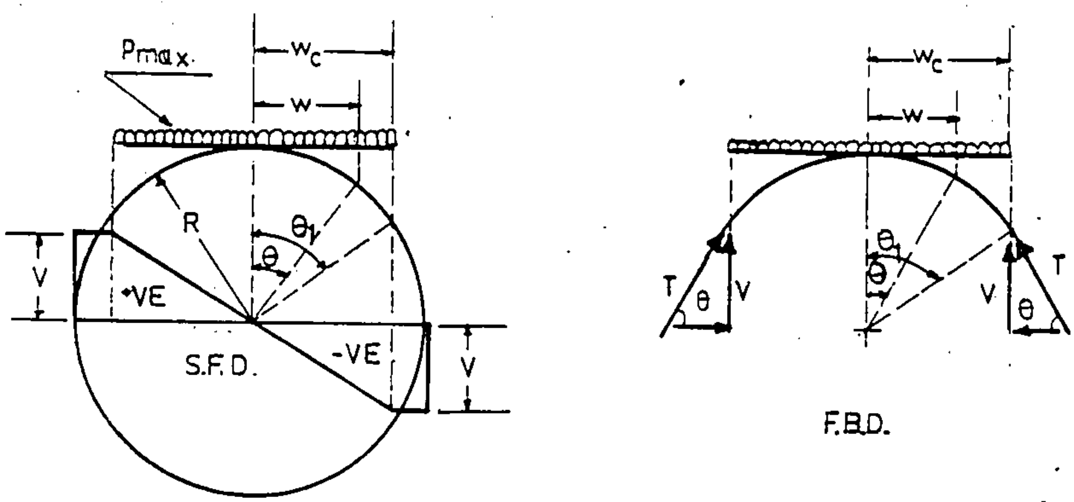


FIG. 3.1a EQUILIBRIUM OF CONDUIT ELEMENT



NEGLECTING MOMENT IN THE CONDUIT

A] $W_c \leq R$

V = SHEAR FORCE AT $\theta\ [0 \leq \theta \leq \sin^{-1}(w_c/R)]$

$= P_{max} * w$

$= P_{max} * R * \sin\theta$ — — — — — — — — — — — Eq. 1

T = AXIAL THRUST

& $V = T * \sin\theta$ — — — — — — — — — — — Eq. 2

Equating Eq. 1 and Eq. 2

$T = P_{max} \cdot R$ For $0 \leq \theta \leq \sin^{-1}[w_c/R]$ — — — Eq. 3

$= P_{max} \cdot w_c / \sin\theta$ For $\sin^{-1}[w_c/R] \leq \theta \leq 90^\circ$ — — Eq. 4

B] $w_c \geq R$

$T = P_{max} \cdot R$ For $0 \leq \theta \leq 90^\circ$ — — — — — — Eq. 5

FIG. 3-1 b PROPOSED THRUST EQUATION

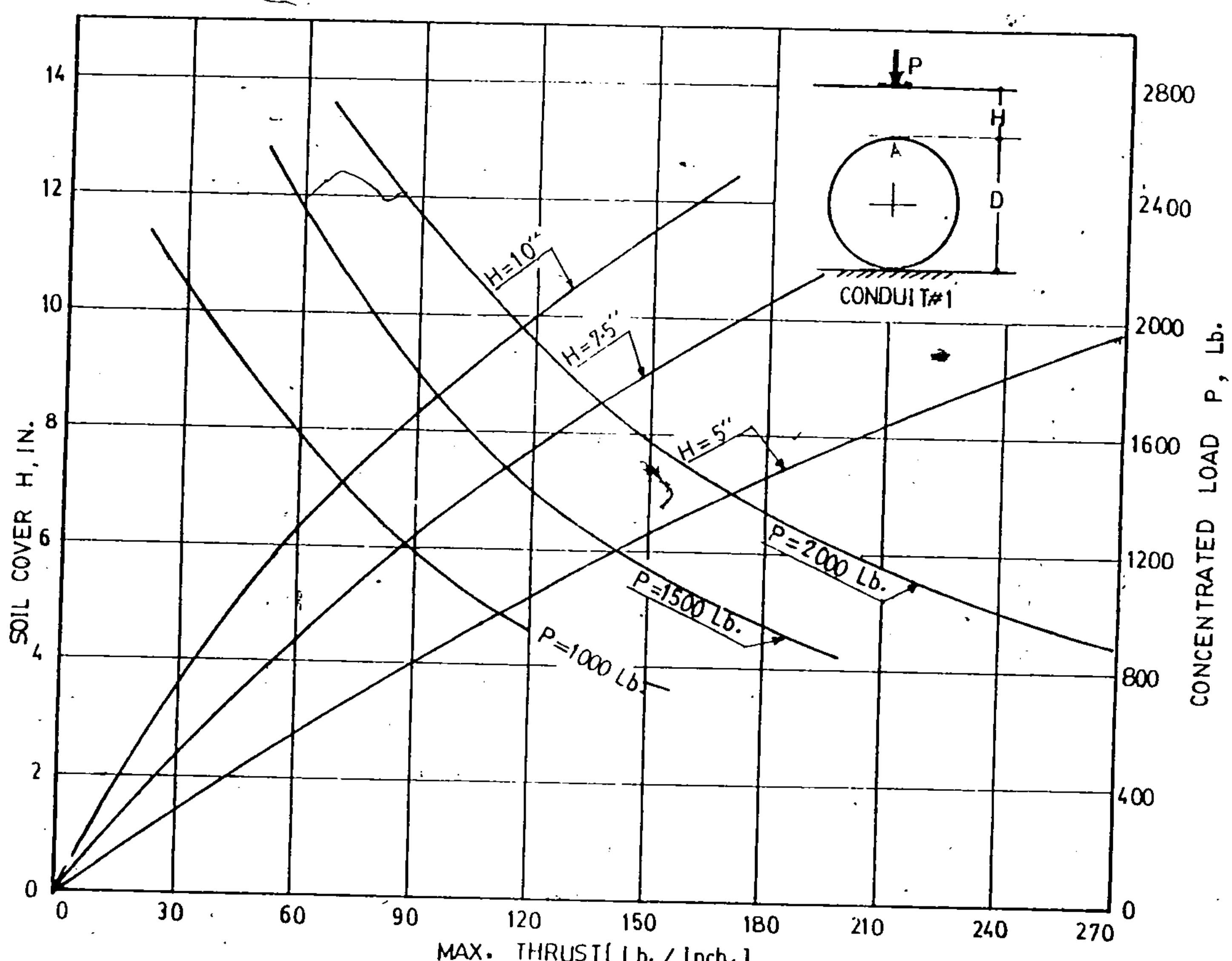


FIG.3.2. EXPERIMENTAL MAXIMUM THRUST VARIATION W.R. TO SOIL COVER AND LOAD - CONDUIT #1

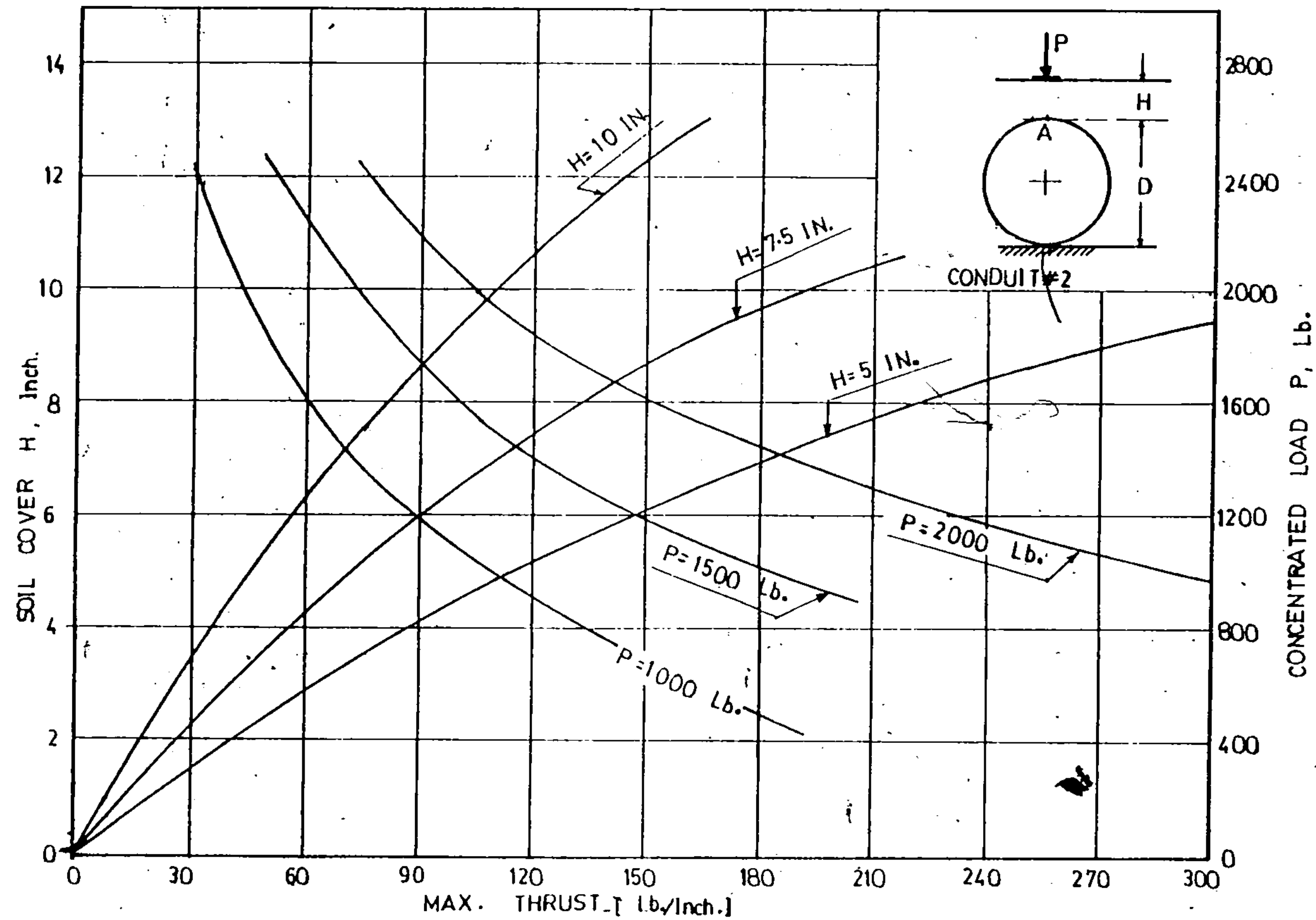


FIG. 3.3. EXPERIMENTAL MAXIMUM THRUST VARIATION W.R. TO SOIL COVER AND LOAD - CONDUIT #2

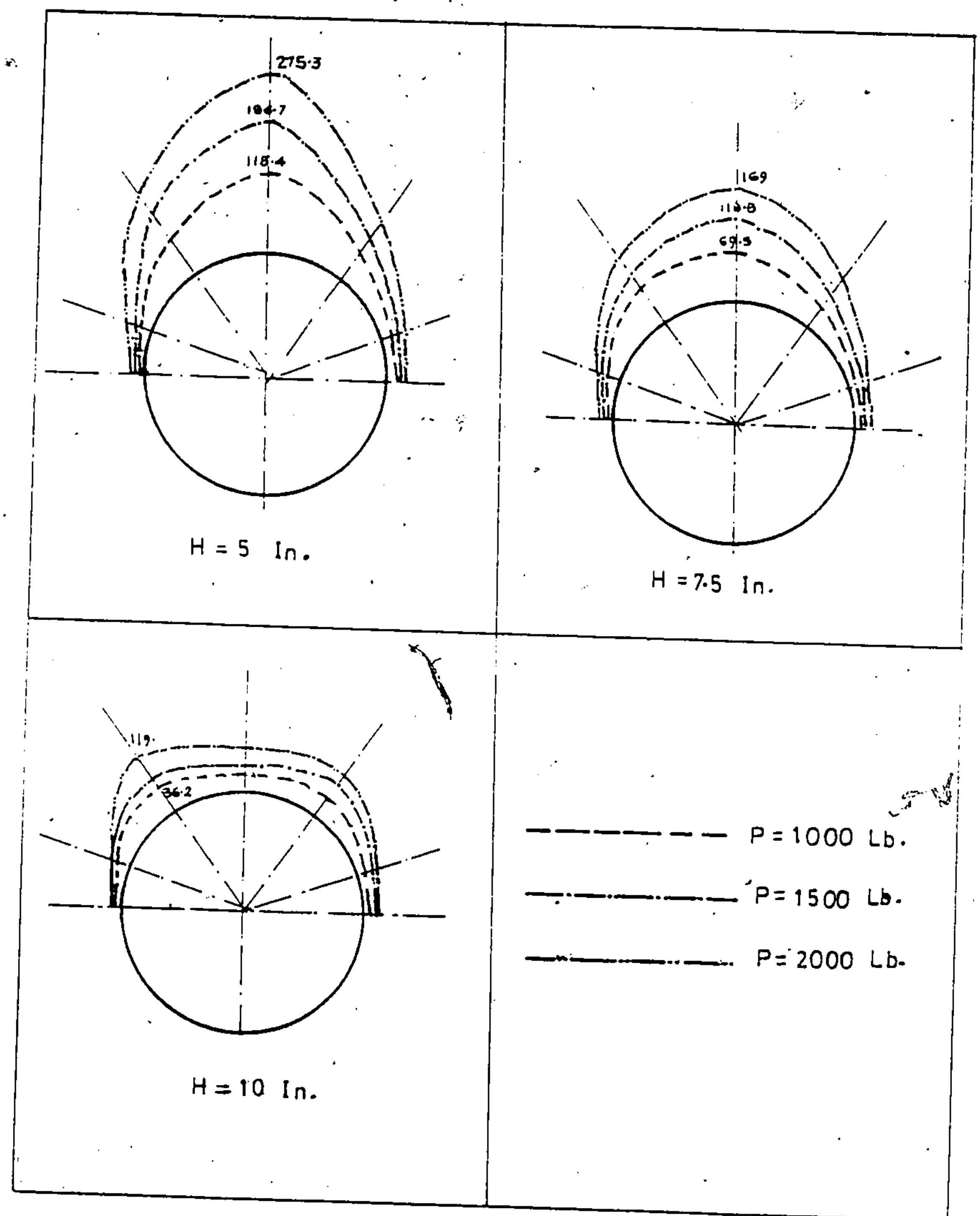


FIG. 3.4 MEASURED THRUST VARIATION – CONDUIT #1

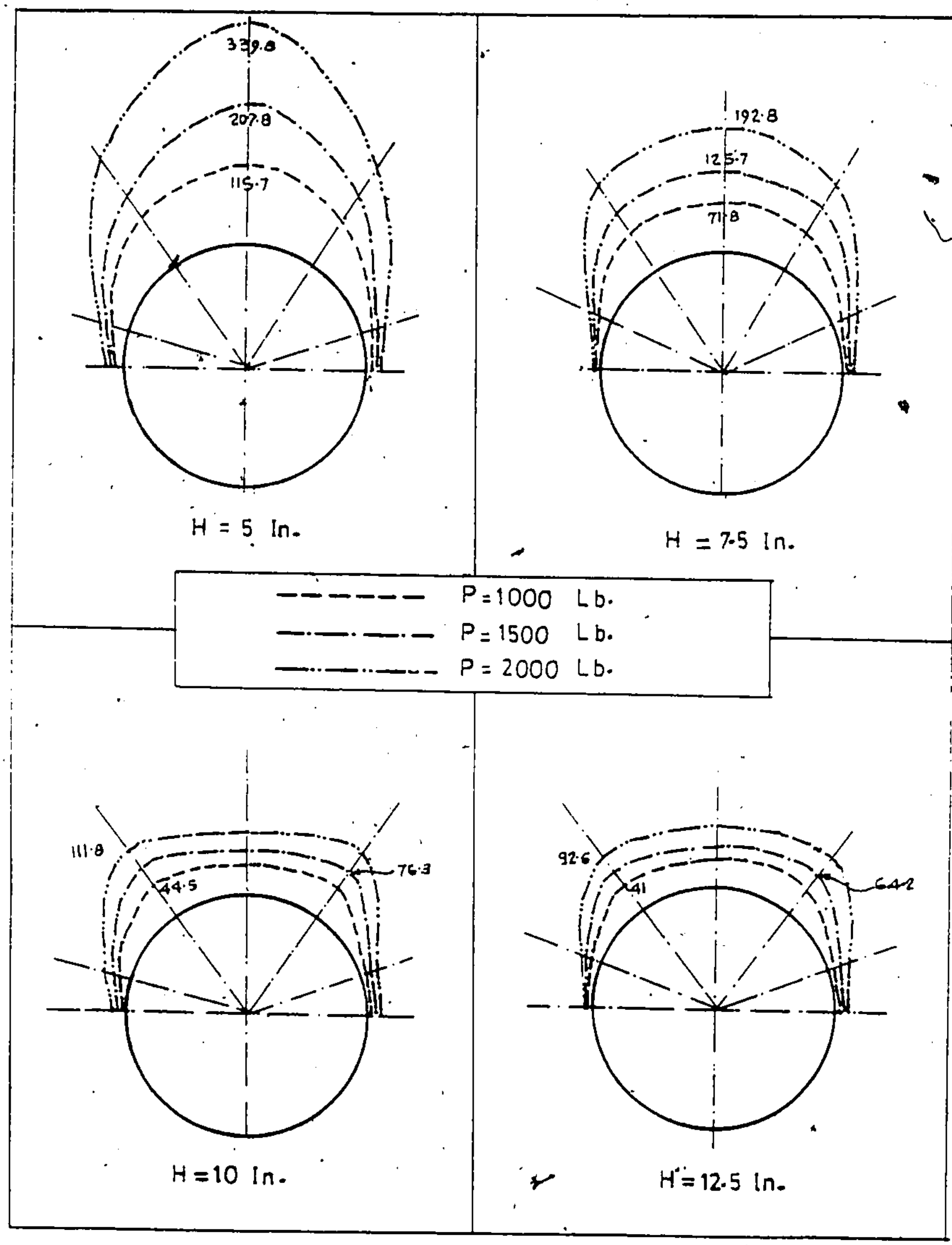


FIG. 3.5 MEASURED THRUST VARIATION - CONDUIT #2

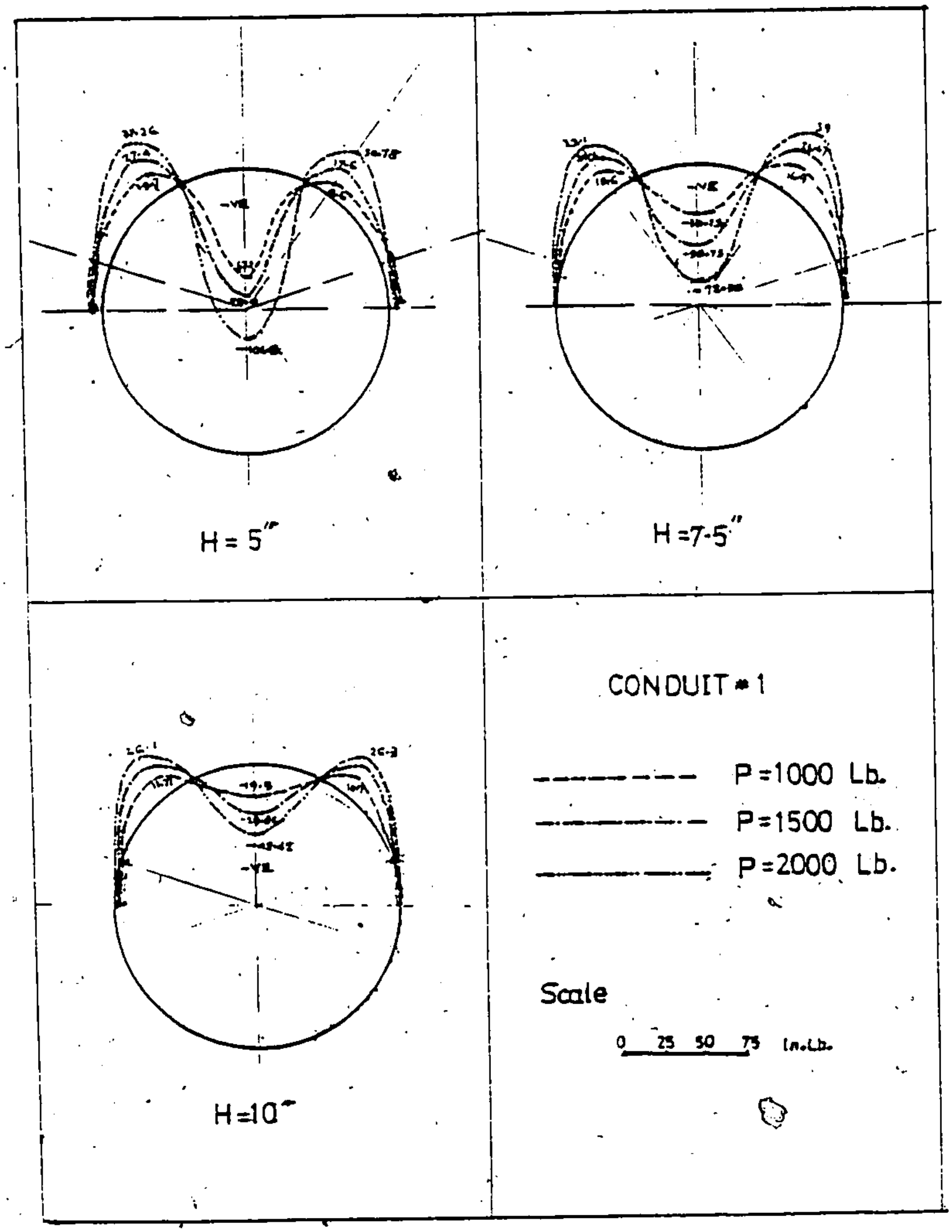


FIG. 3.6 BENDING MOMENT VARIATION - CONDUIT # 1

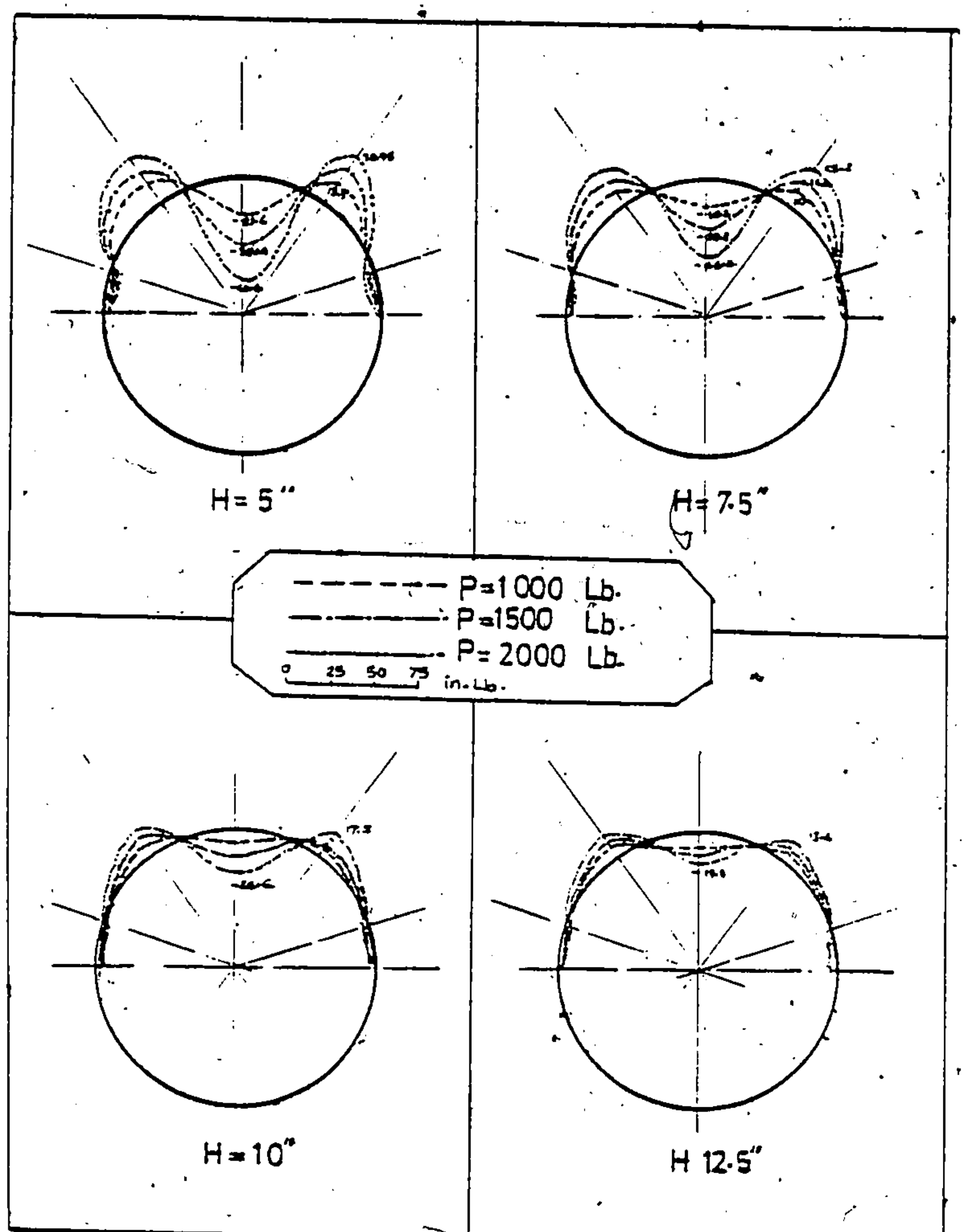


FIG. 3.7 BENDING MOMENT VARIATION - CONDUIT #2

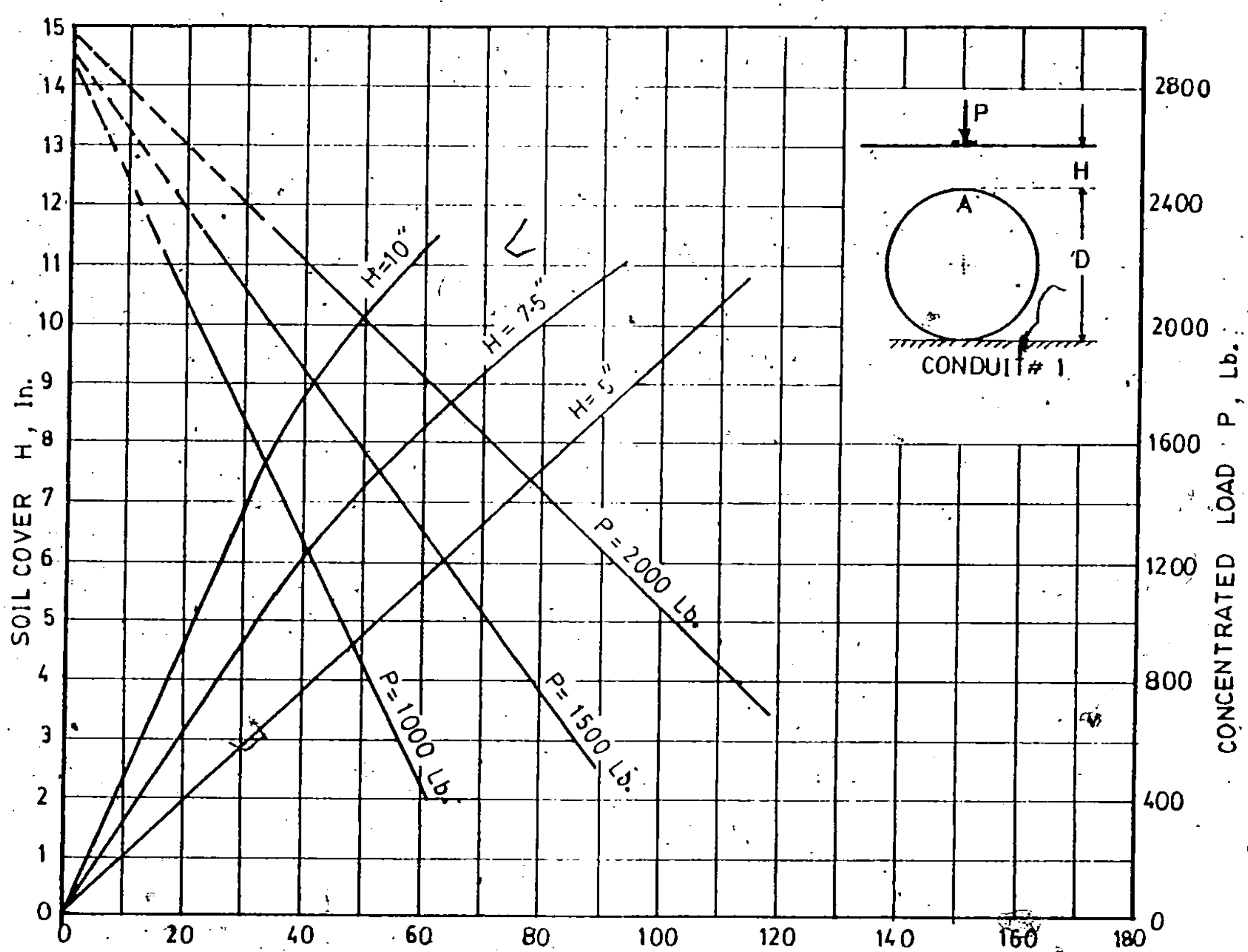


FIG. 3.8 MEASURED BENDING MOMENT (M_a) VARIATION W.R. TO SOIL COVER AND LOAD - CONDUIT#1

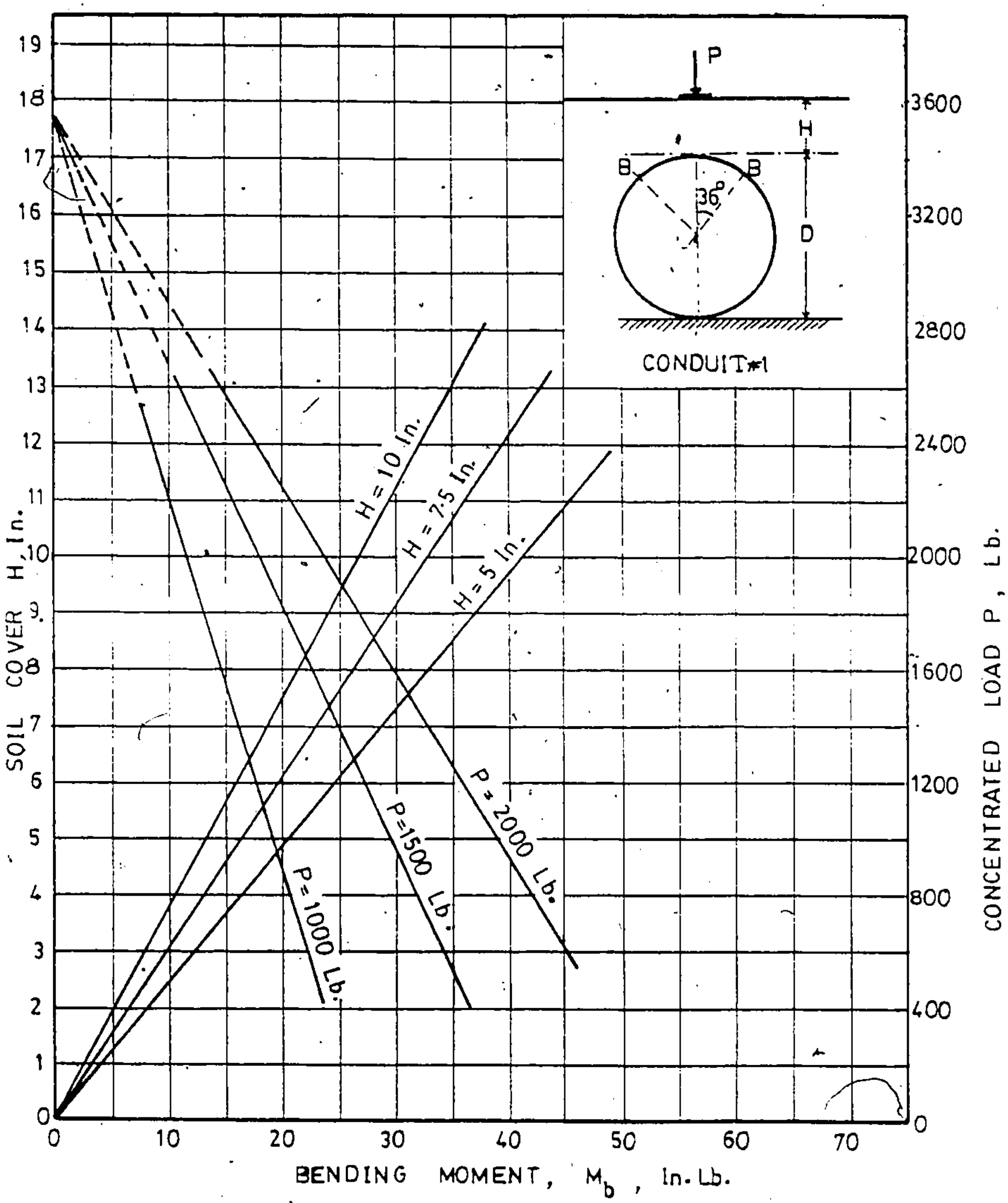


FIG. 3-9 MEASURED BENDING MOMENT (Mb) VARIATION W.R. TO SOIL COVER AND LOAD CONDUIT #1

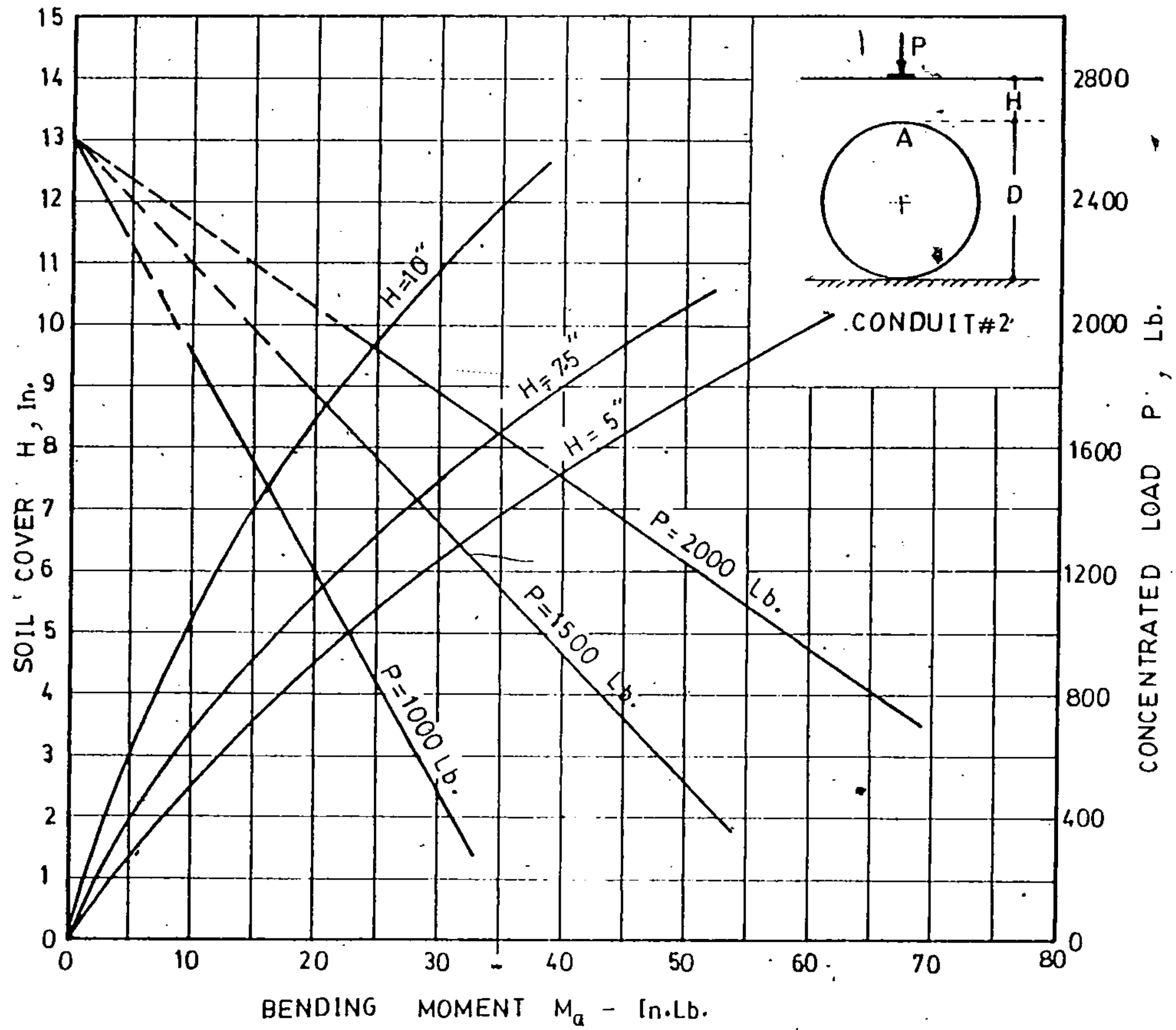


FIG.3.10 MEASURED BENDING MOMENT (Ma) VARIATION W.R. TO SOIL COVER AND LOAD - CONDUIT #2

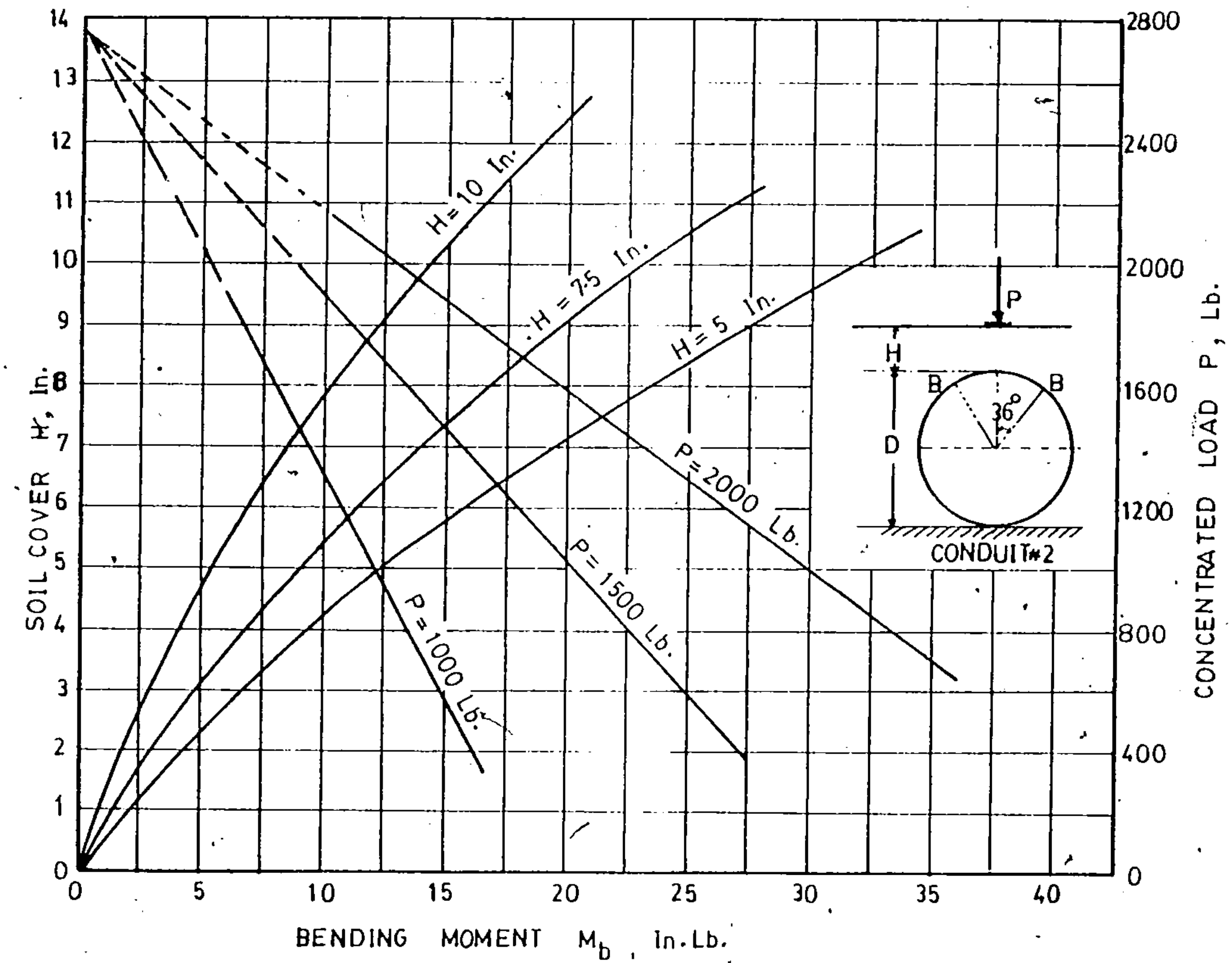


FIG. 3.11 MEASURED BENDING MOMENT (M_b) VARIATION W.R. TO SOIL COVER AND LOAD - CONDUIT #2

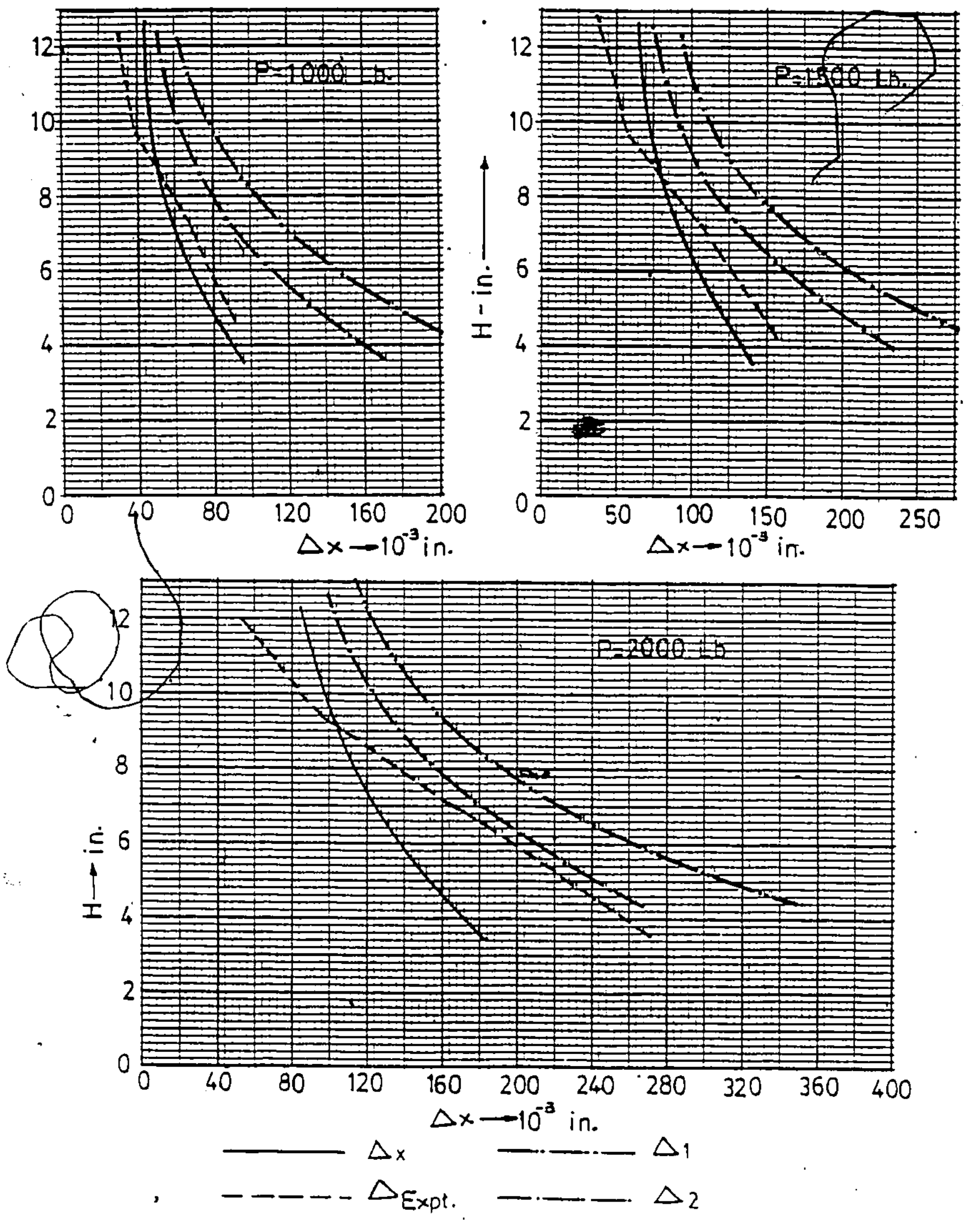


FIG. 3.12 Variation of Deflection w. r. to Soil Cover ---H

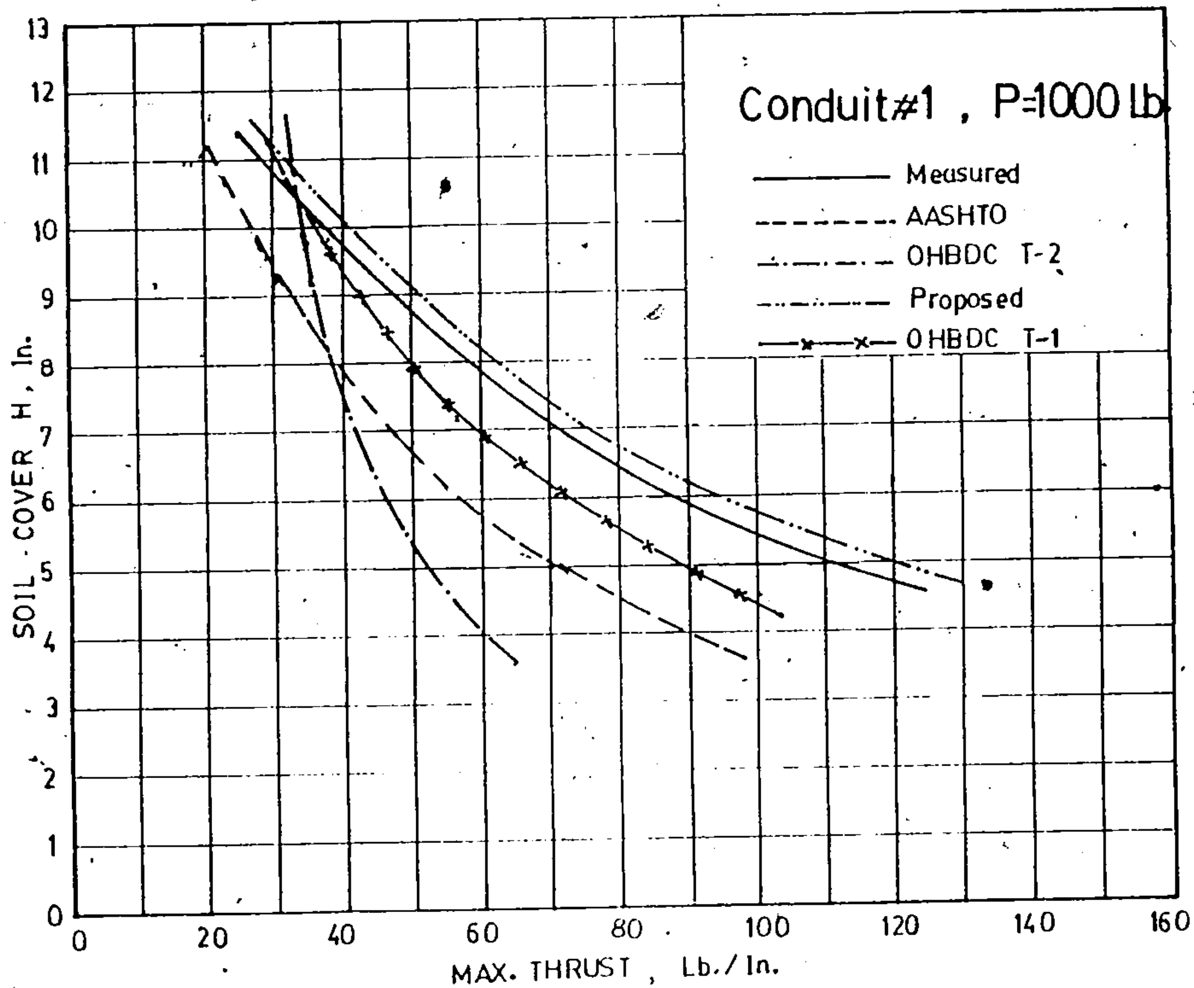


FIG. 4.1 COMPARISON OF MAX. AXIAL THRUST OBTAINED BY VARIOUS METHODS
(FOR, P=1000 lb. & CONDUIT #1)

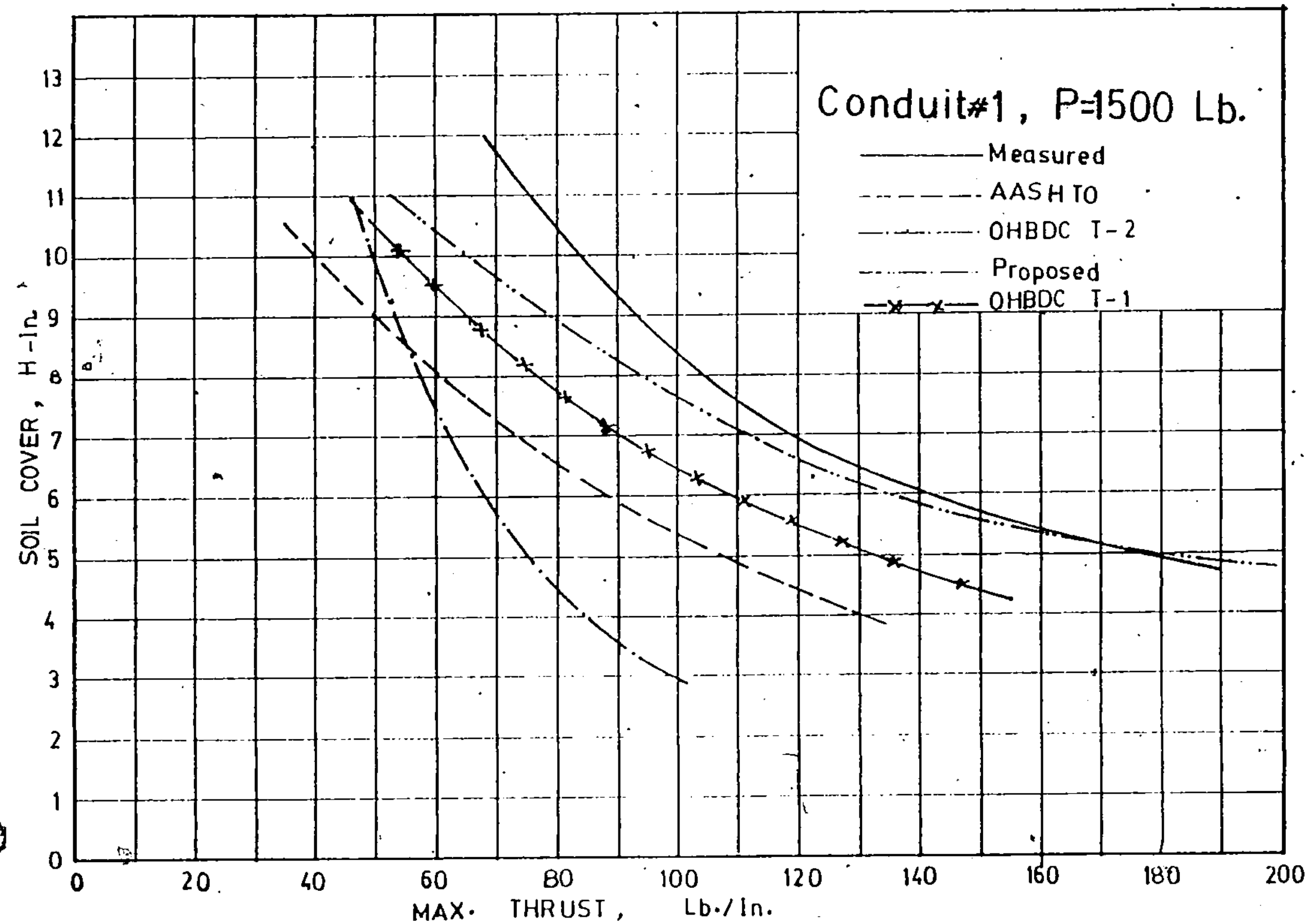


FIG. 4.2 COMPARISON OF MAX. AXIAL THRUST OBTAINED BY VARIOUS METHODS

(FOR P=1500 Lb & CONDUIT #1)

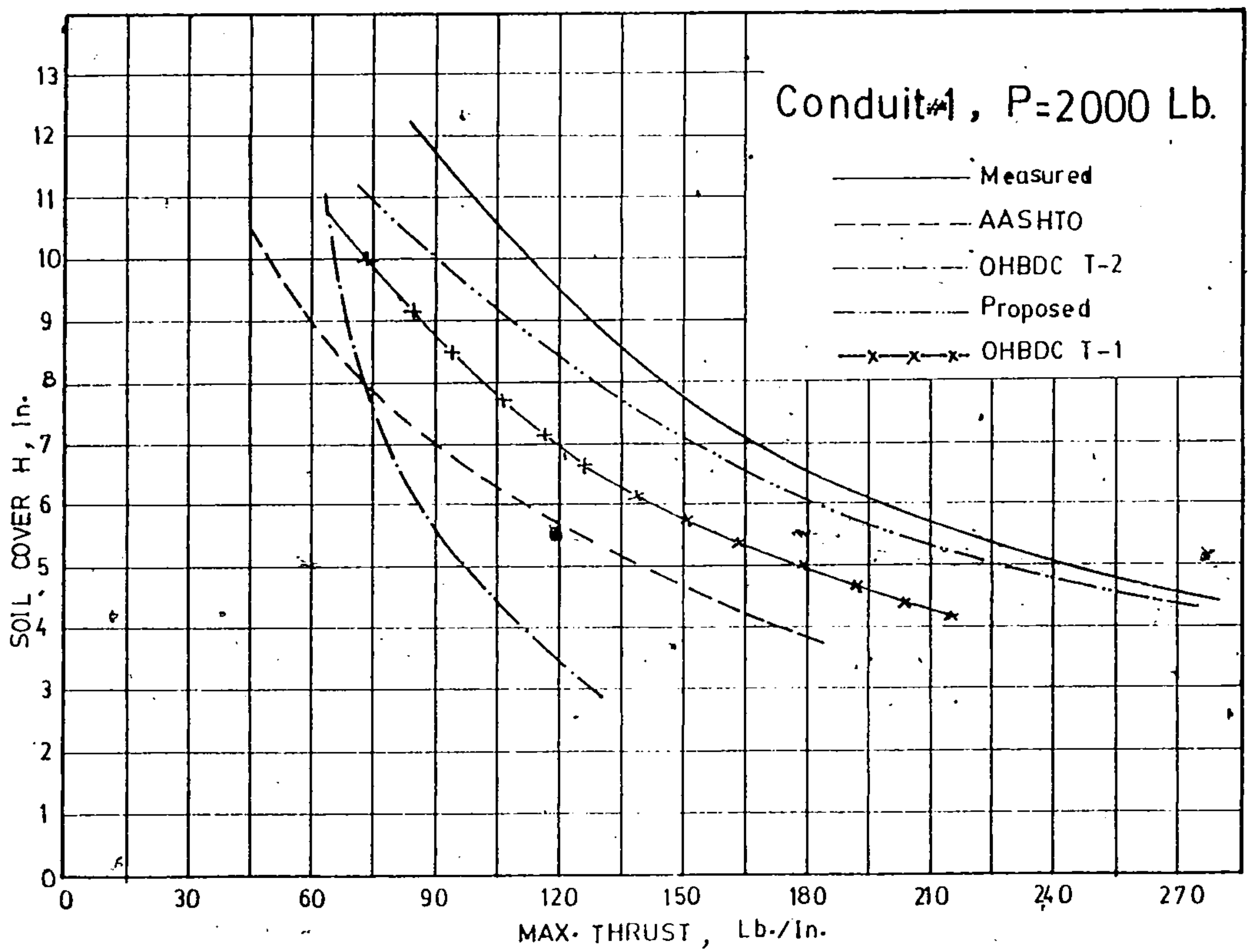


FIG. 4.3 COMPARISON OF MAX. AXIAL THRUST OBTAINED BY VARIOUS METHODS

(FOR P=2000 lb. AND CONDUIT # 1)

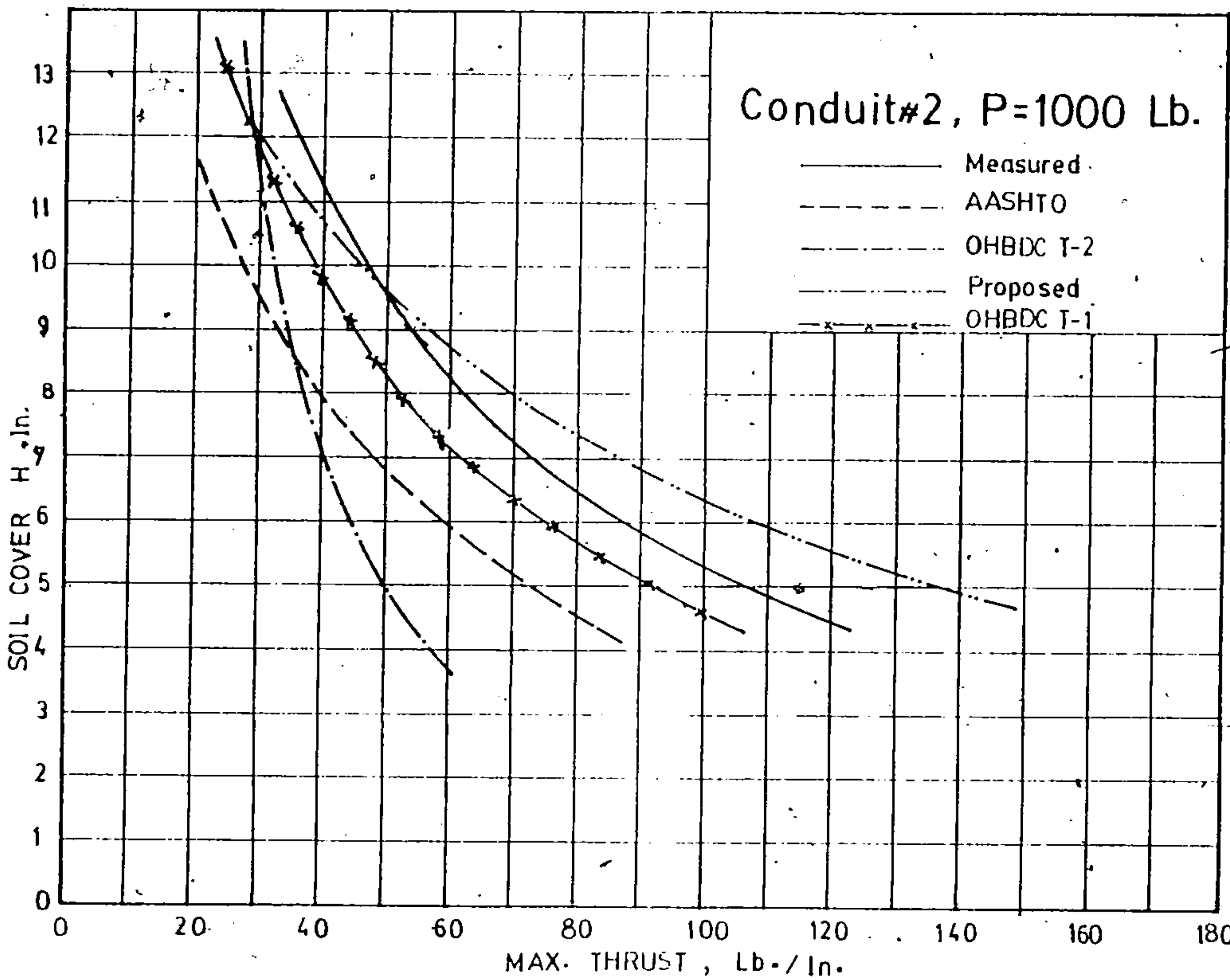


FIG.4.4. COMPARISON OF MAX. AXIAL THRUST OBTAINED BY VARIOUS METHODS

(FOR P =1000 lb. & CONDUIT #2)

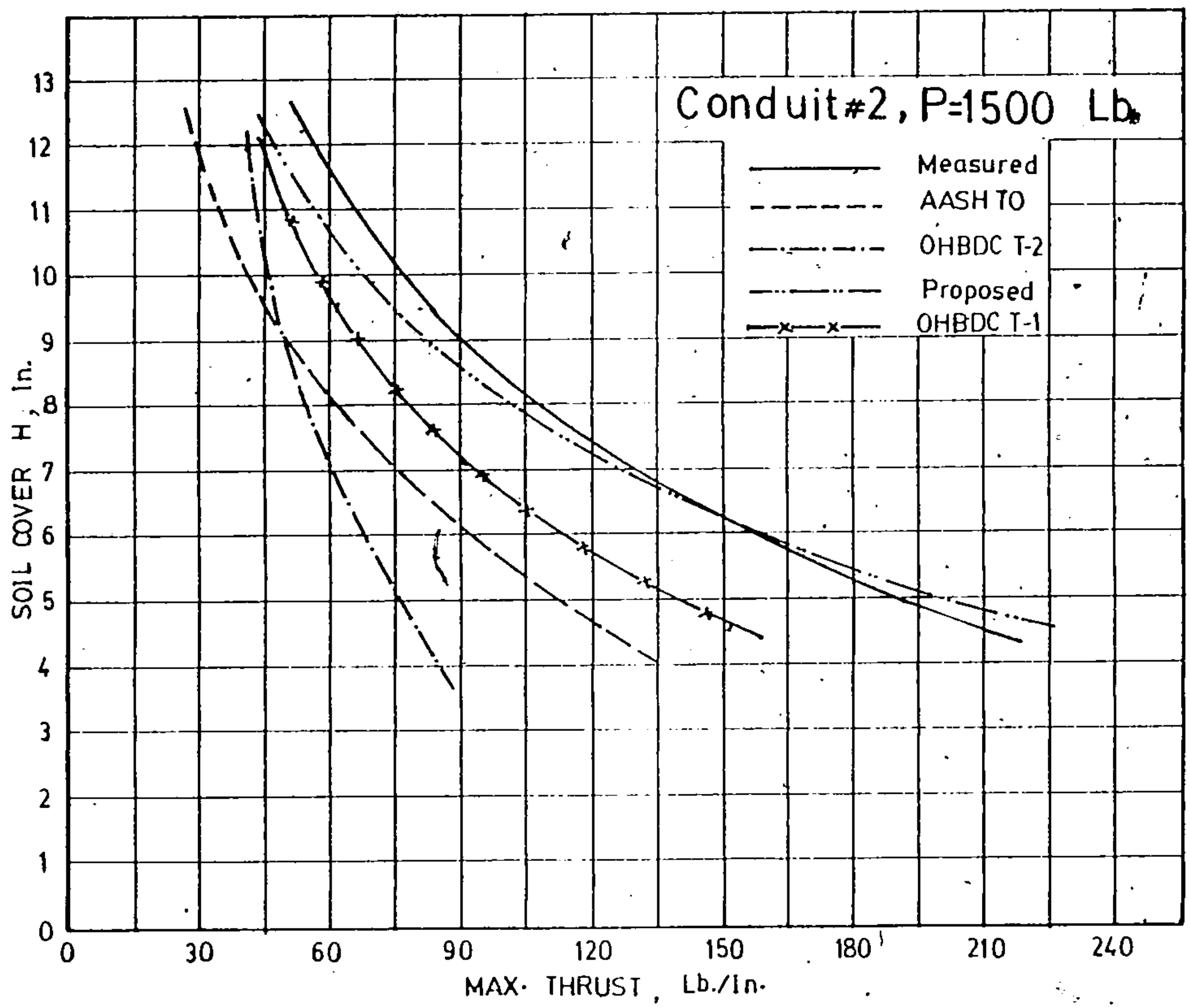


FIG.4.5 COMPARISON OF MAX. AXIAL THRUST OBTAINED BY VARIOUS METHODS

(FOR P =1500 lb. & CONDUIT #2)

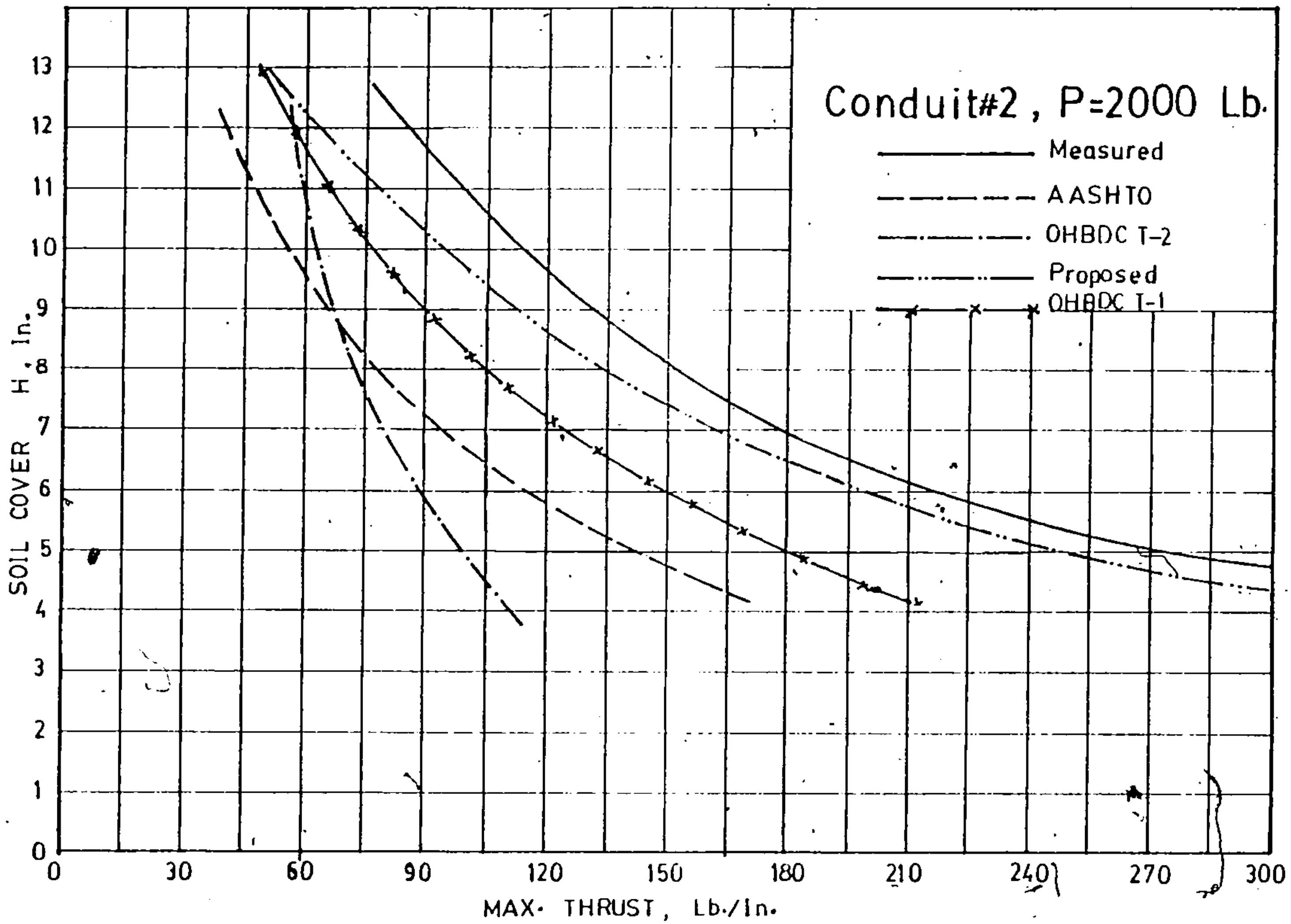


FIG.4.6 COMPARISON OF MAX. AXIAL THRUST OBTAINED BY VARIOUS METHODS

(FOR P =2000 lb. & CONDUIT #2)

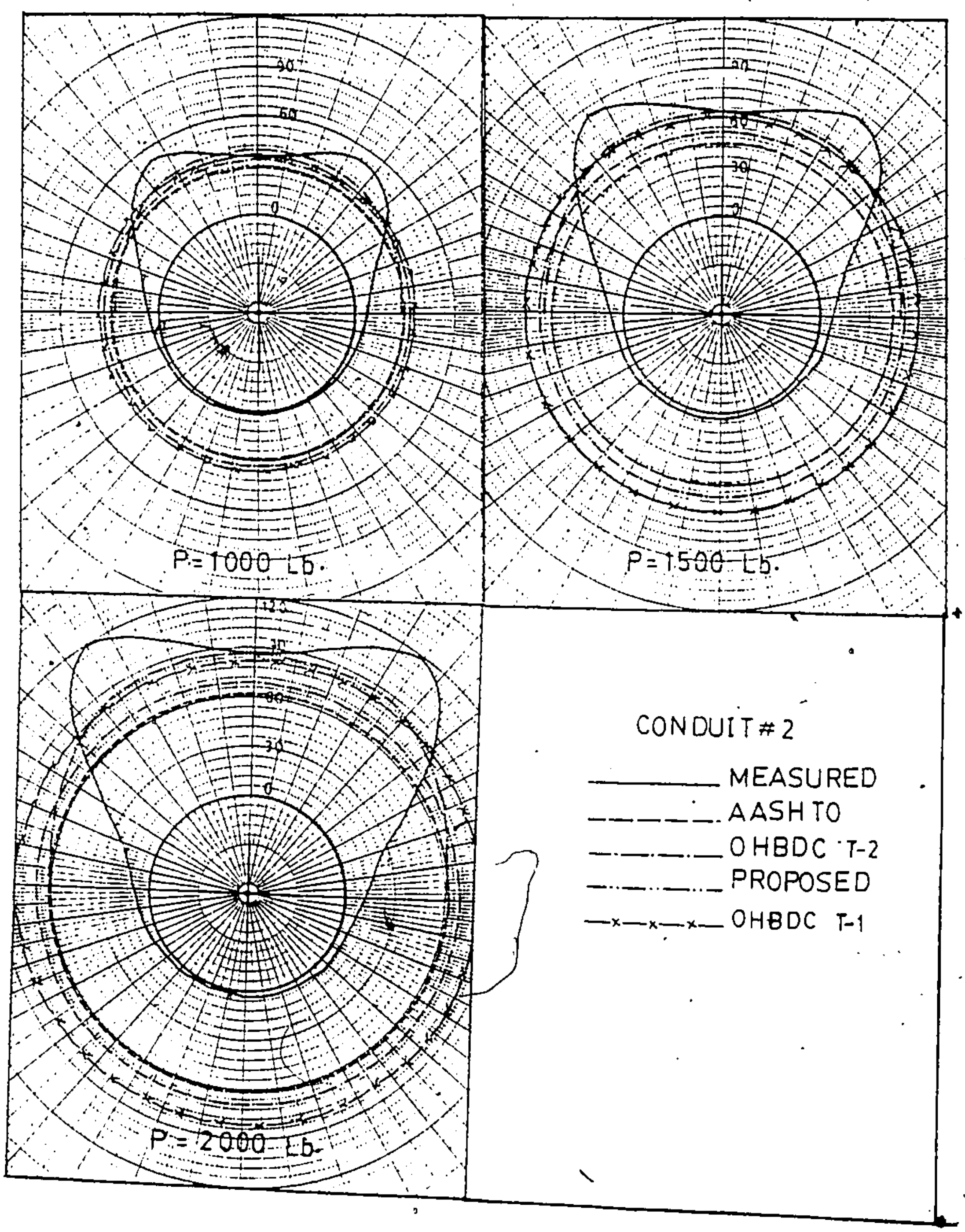
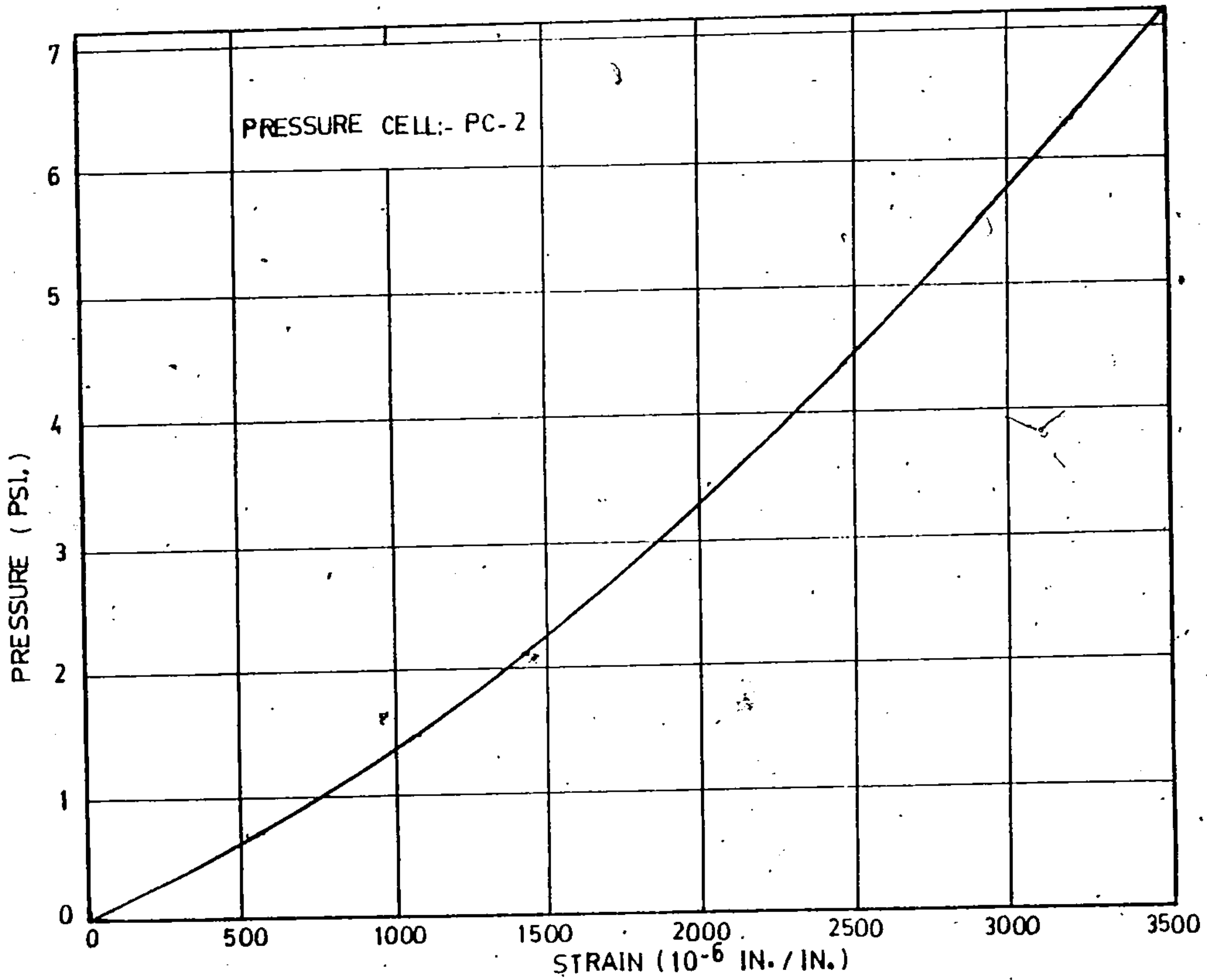


FIG. 4.7 THRUST BY VARIOUS METHODS FOR H = 10 INCH.

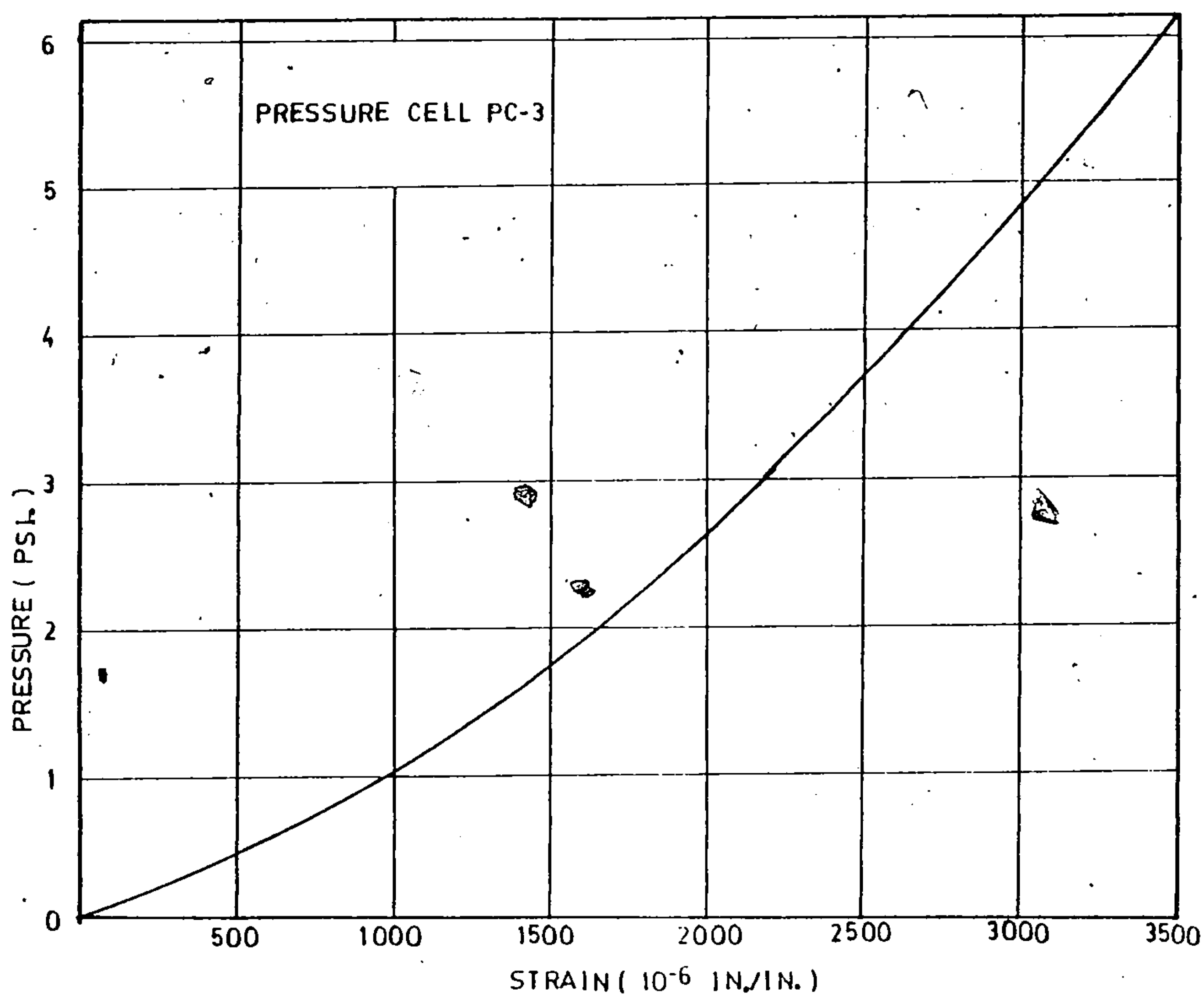
APPENDIX: A

CALIBRATION CHARTS FOR PRESSURE CELLS.



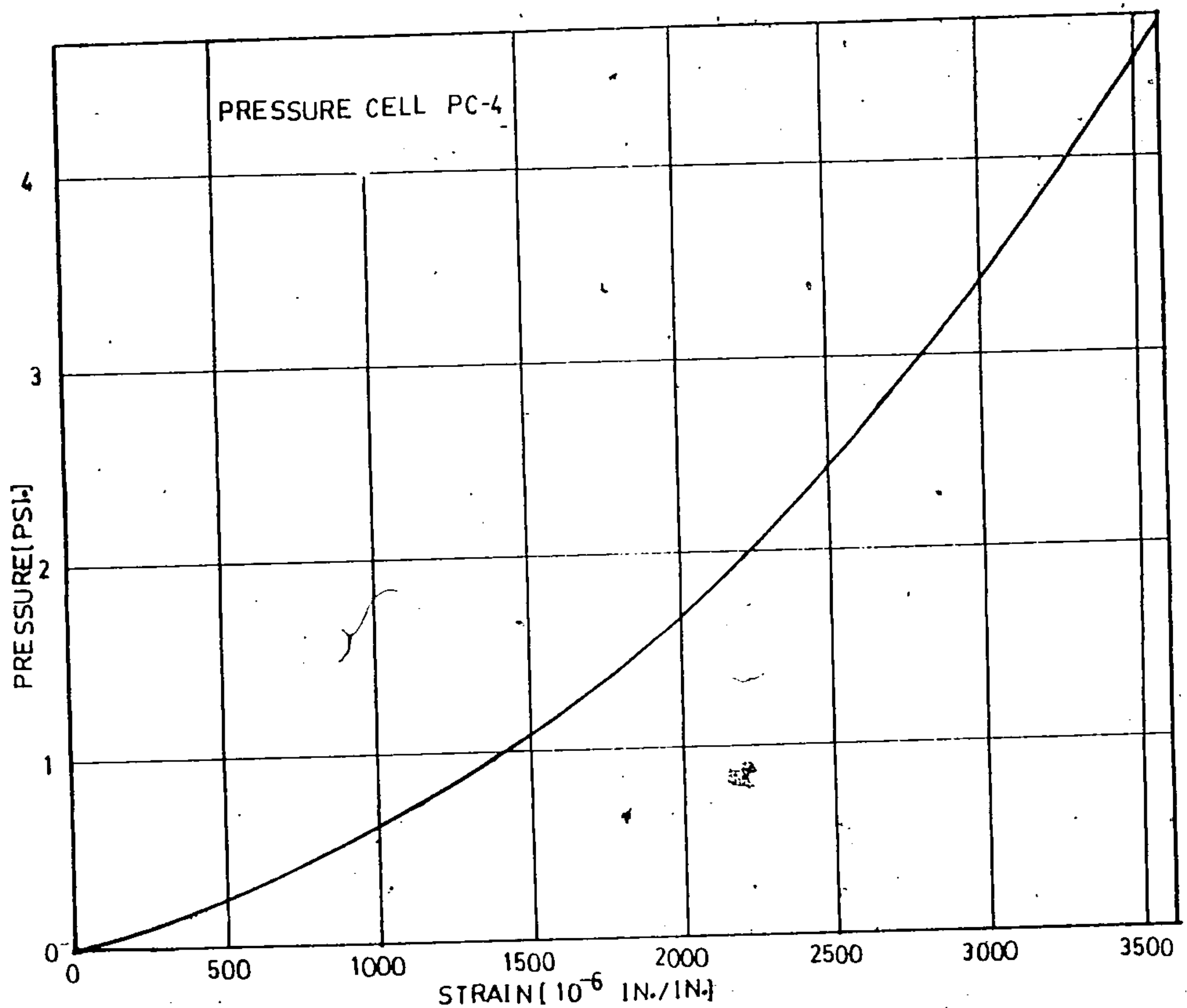
APPENDIX-A :- CALIBRATION CHART FOR PRESSURE

CELL PC - 2



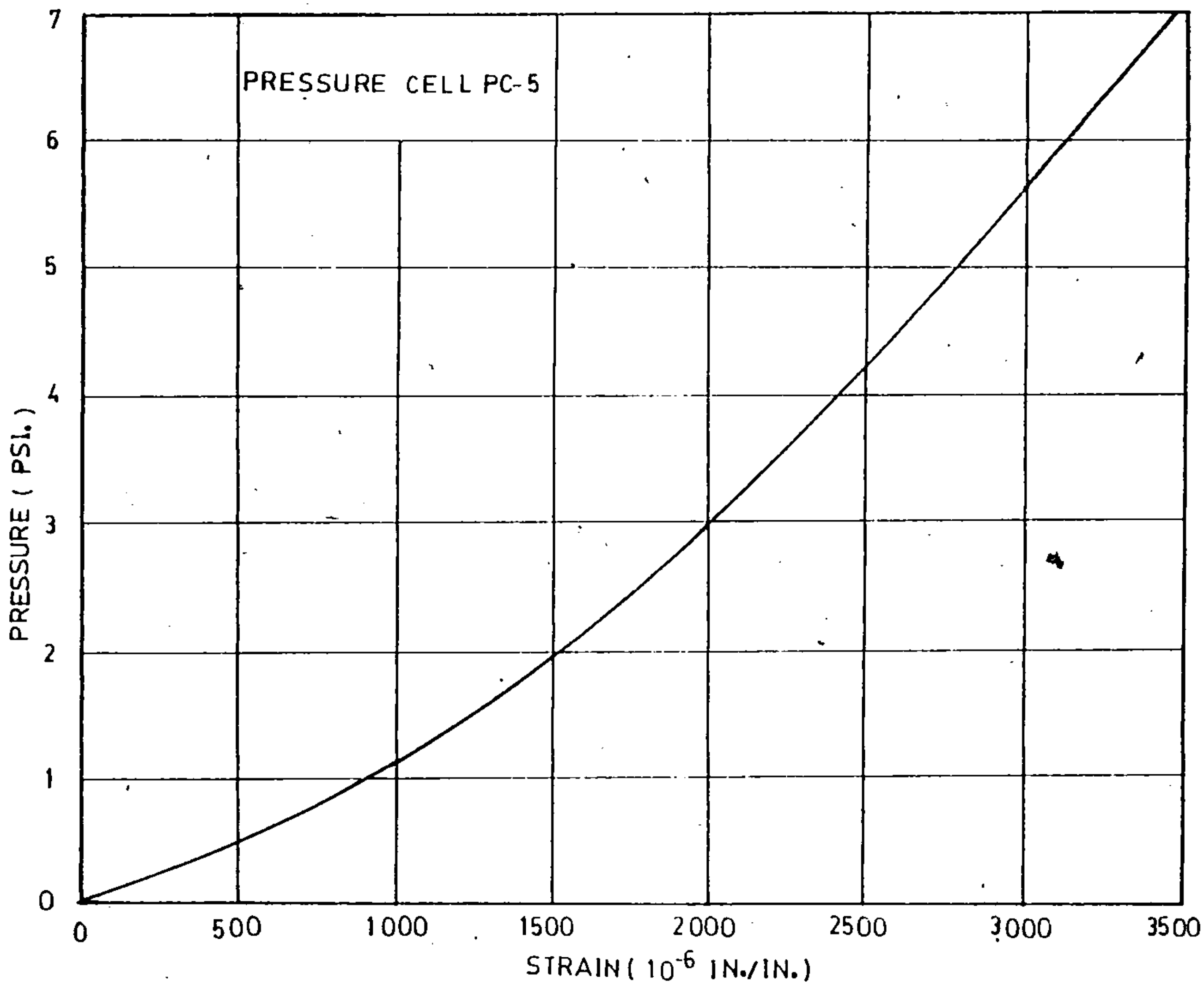
APPENDIX -A :- CALIBRATION CHART FOR PRESSURE

CELL PC - 3

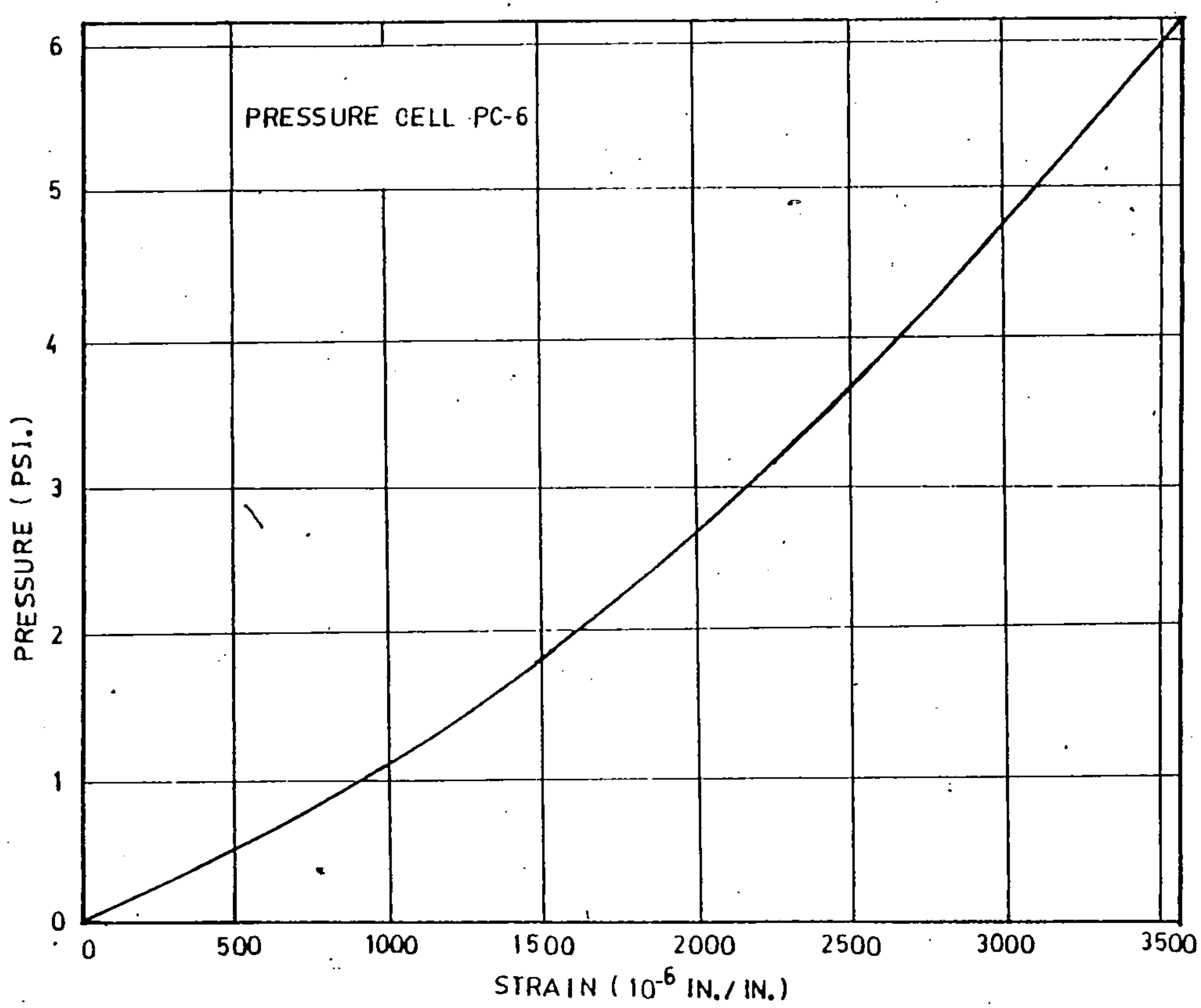


APPENDIX – A :- CALIBRATION CHART FOR PRESSURE

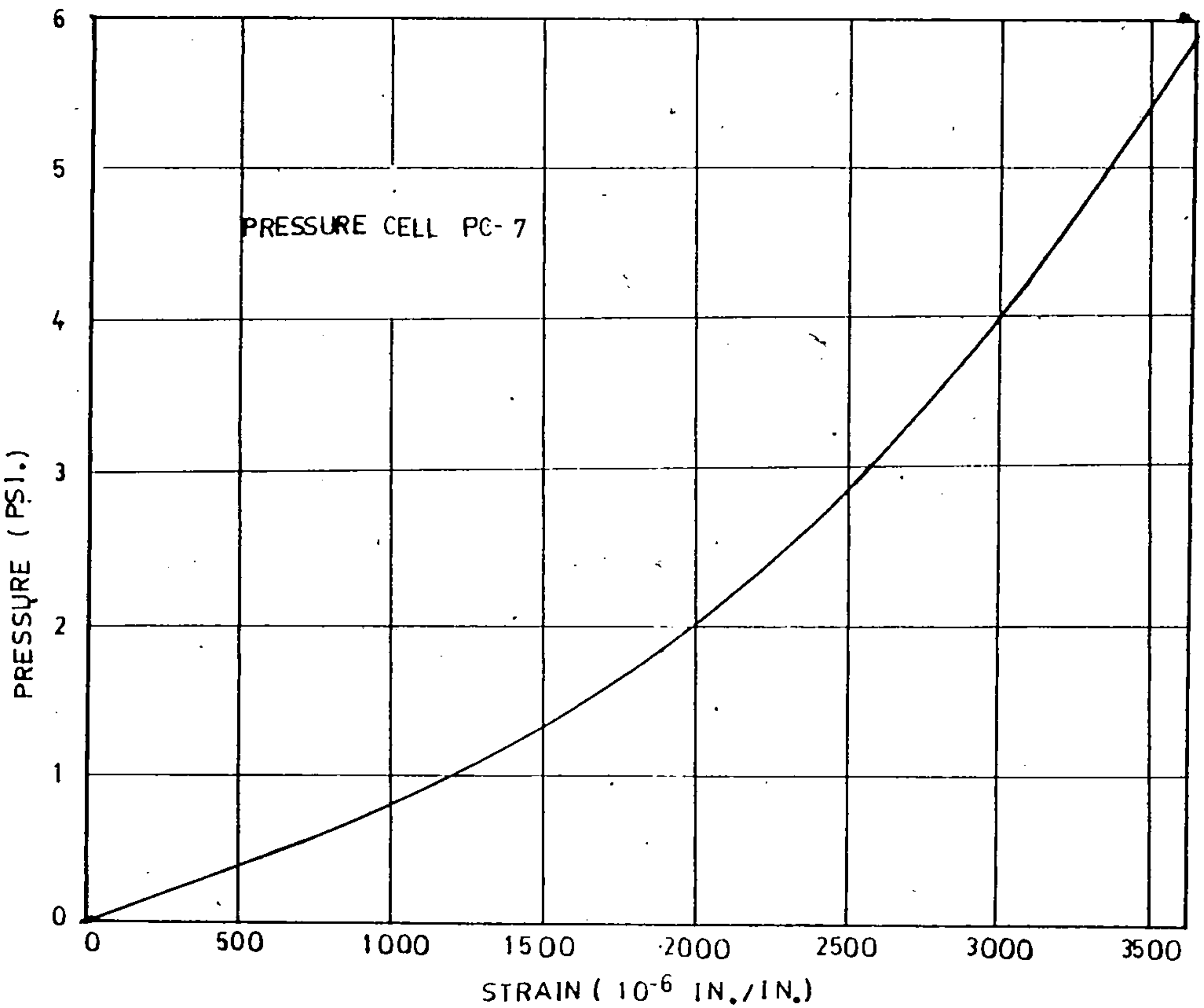
CELL PC-4



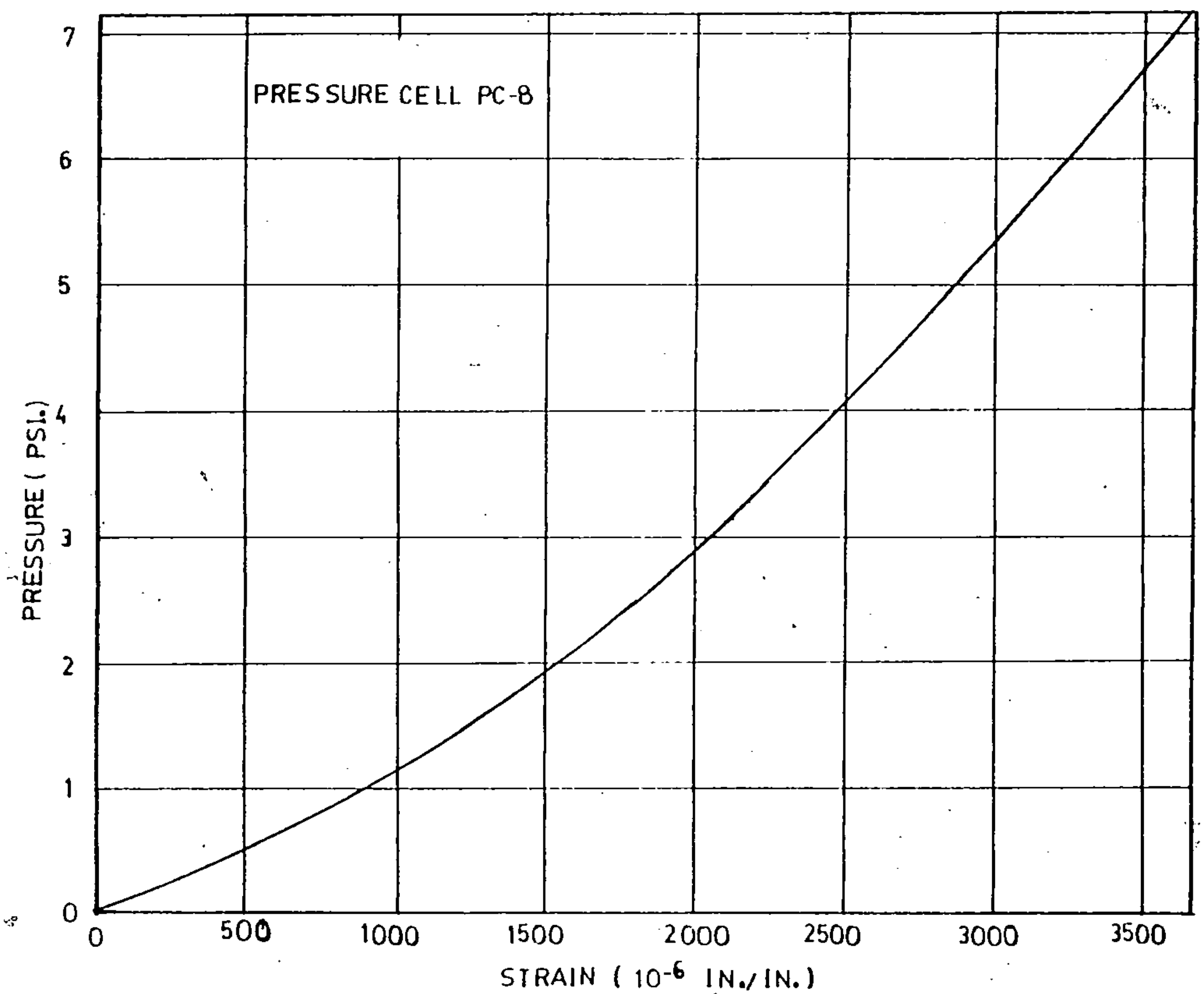
APPENDIX-A :- CALIBRATION CHART FOR PRESSURE CELL, PC-5



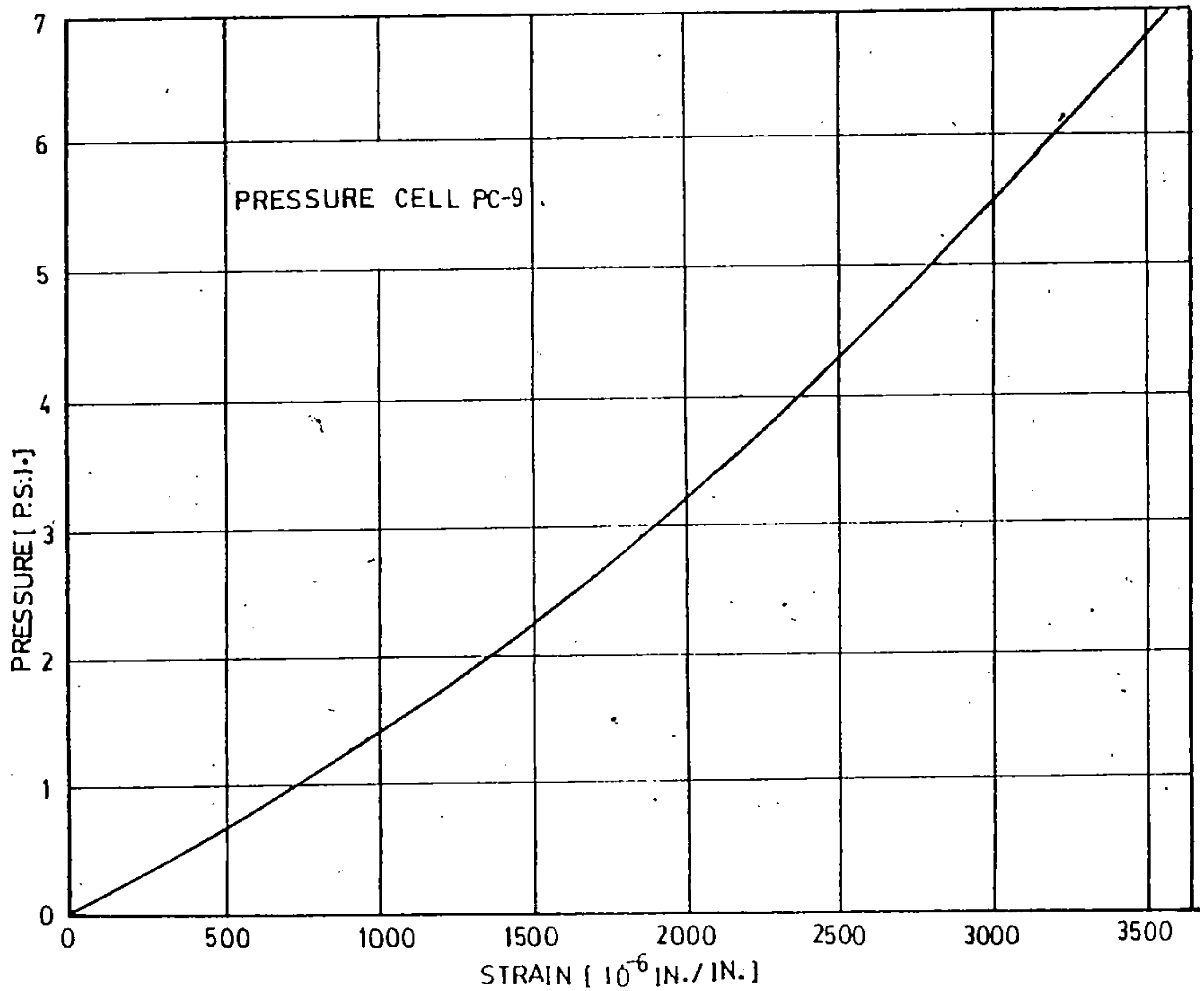
APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-6



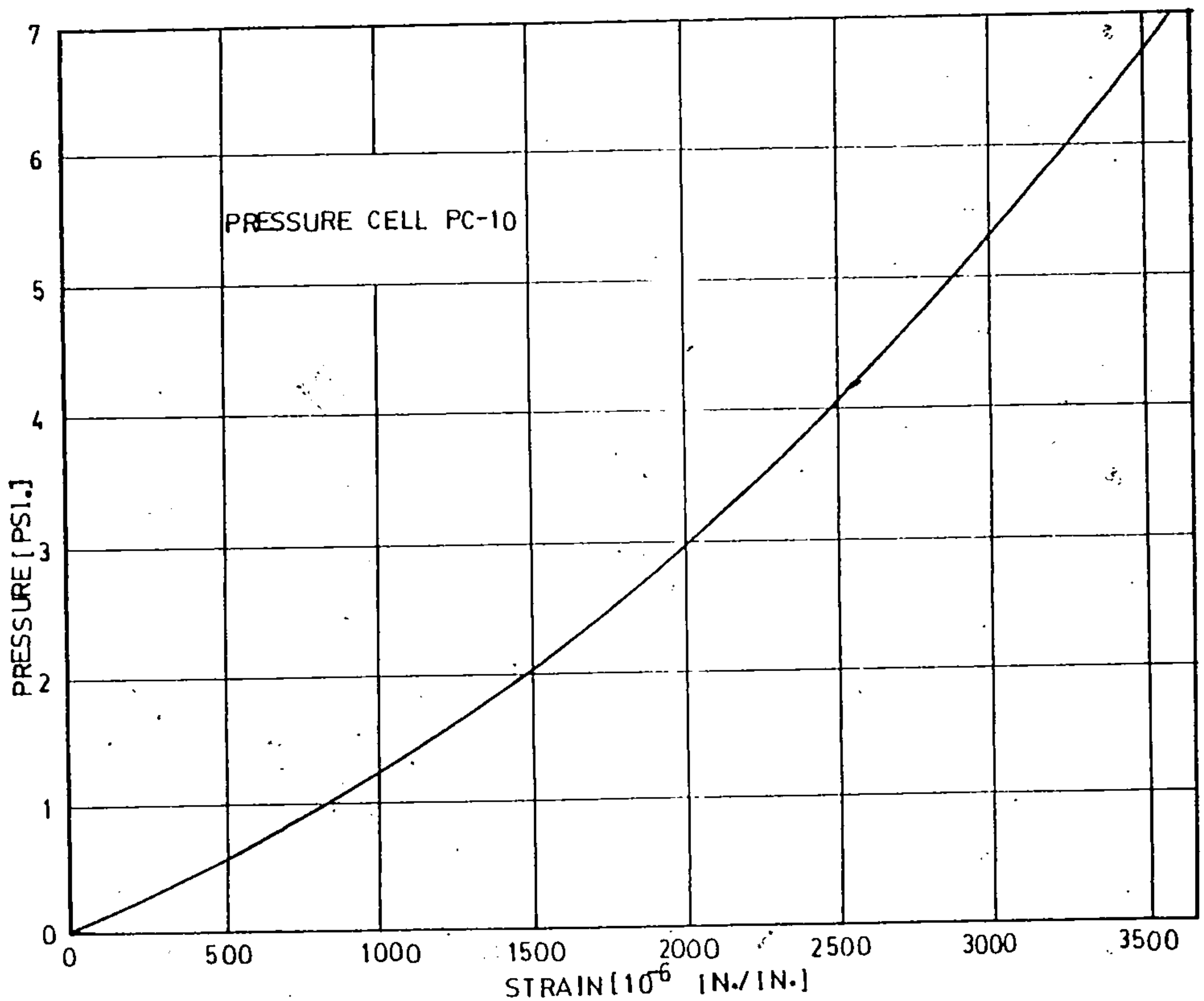
APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-7.



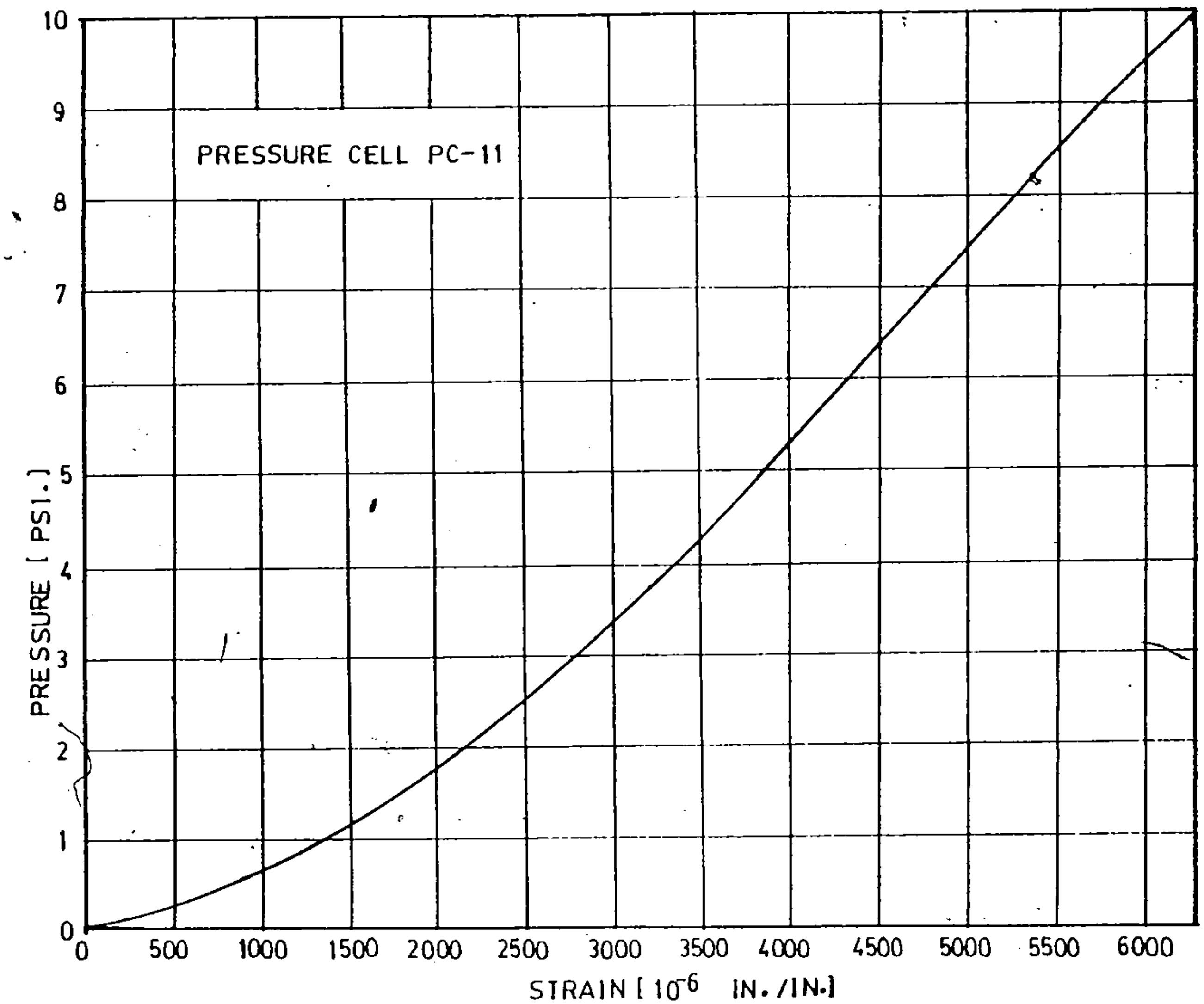
APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-8



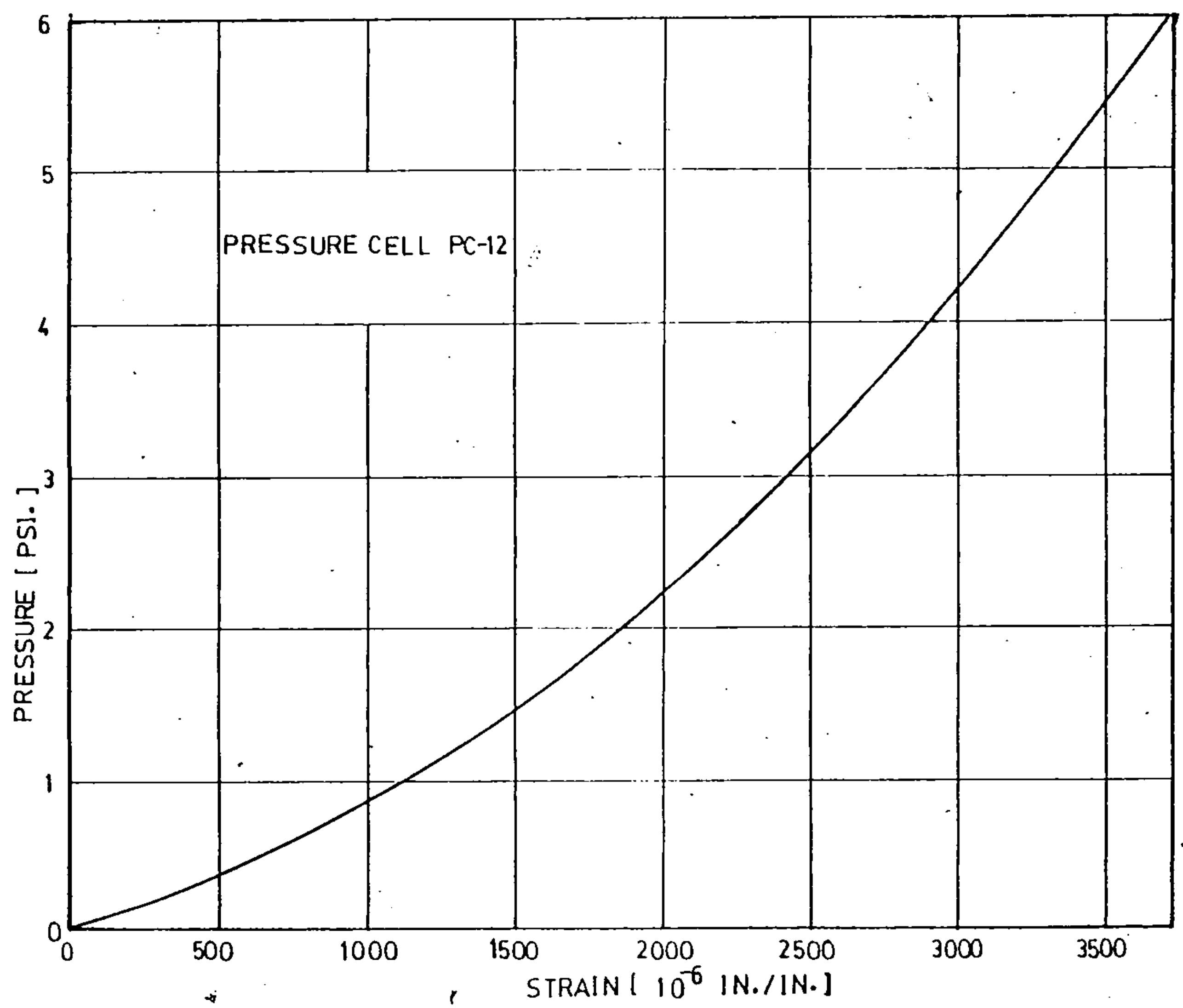
APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-9



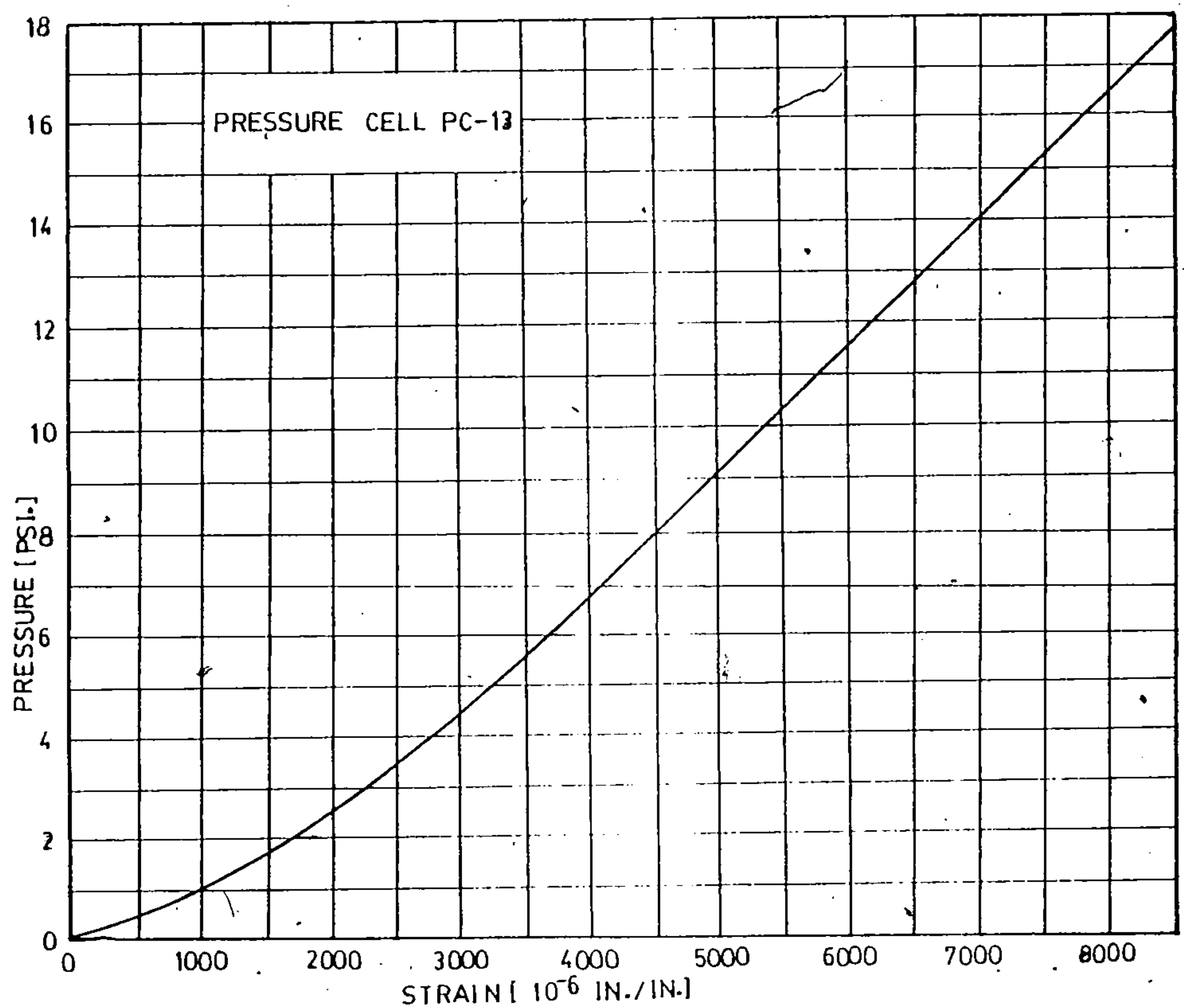
APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-10



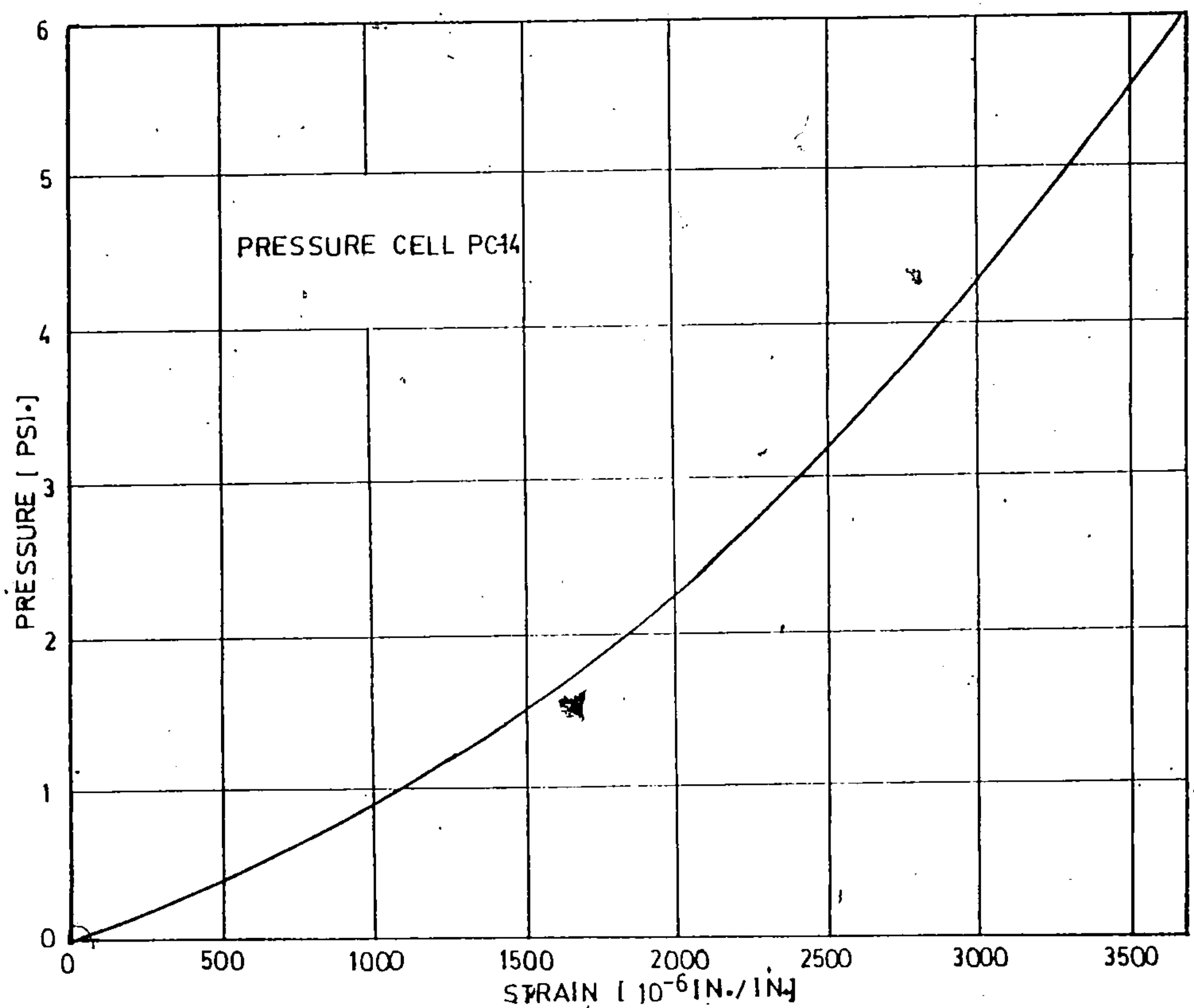
APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-11



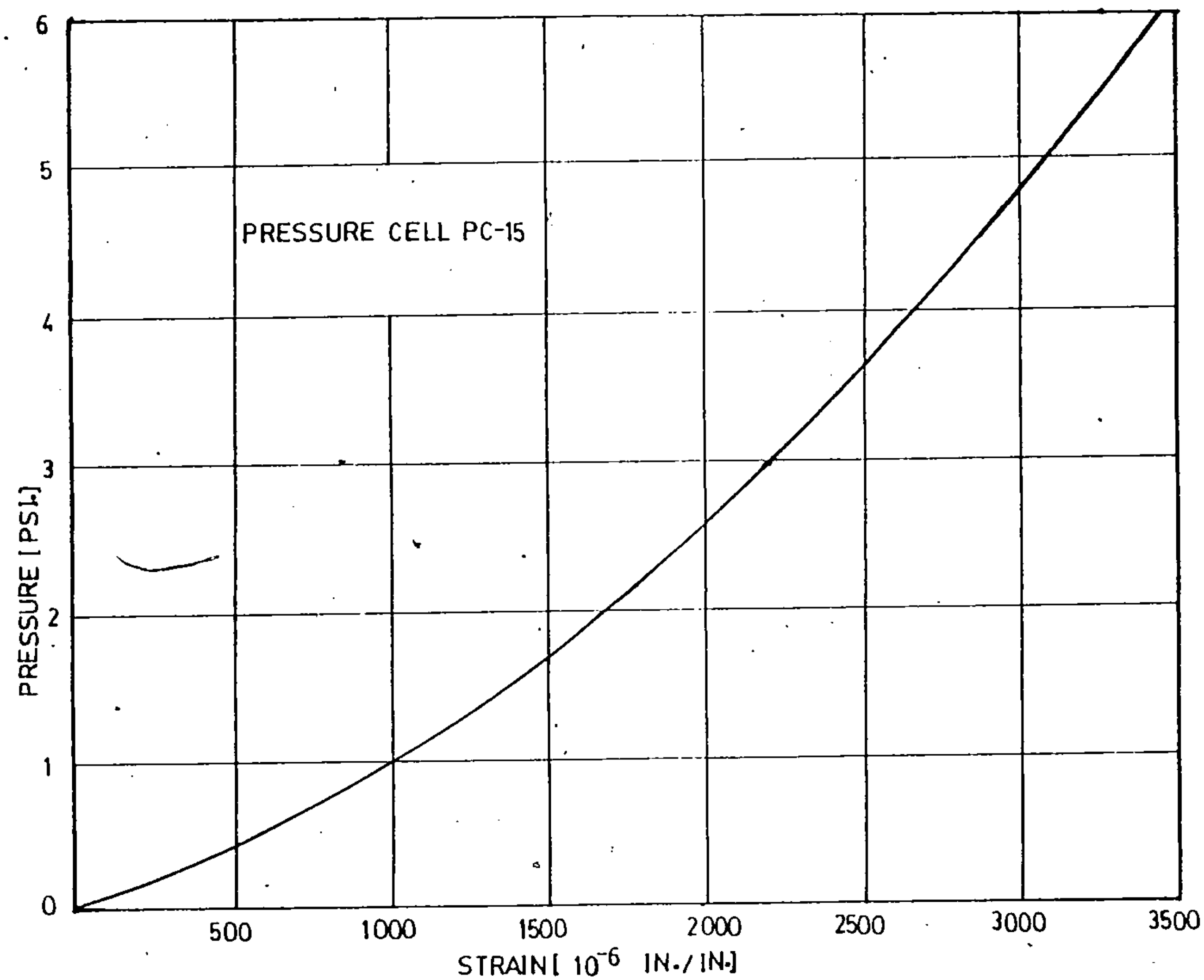
APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-12



APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-13



APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-14



APPENDIX A :- CALIBRATION CHART FOR PRESSURE CELL, PC-15

APPENDIX :- B

COMPUTER PROGRAMS.

```
$JOB    WATFIV   XXXXXXXXXXS   EKHANDE
C      --------------------------------------------------------------
C      -                                                            -
C      -                  SHANTARAM G.EKHANDE                       -
C      -                                                            -
C      --------------------------------------------------------------
C      -                                                            -
C      -                  SOIL-STEEL STRUCTURE                      -
C      -                                                            -
C      --------------------------------------------------------------
C    ******************************************************************
C    *  THIS PROGRAM CALCULATES THRUST & BENDING MOMENT IN A CONDUIT  *
C    *  MODEL TESTED FOR DIFFERENT SOIL COVERS,DIFFERENT MAGNITUDE OF *
C    *  CONCENTATED LOADS & DIFFERENT ECCENTRICITY OF LOADS.          *
C    *  DSIGMA=AXIAL STRESS IN PSI.                                   *
C    *  BSIGMA=BENDING STRESS IN PSI.                                 *
C    *  TYPE OF STRAIN GAGES USED :- CEA-13-500UW-120                 *
C    *  GAGE FACTOR USED :- 2.090  +/- 0.5% AT 75 F                   *
C    *  NO. OF STRAIN GAGES USED=44. OF WHICH 22 ON OUTSIDE & 22      *
C    *  INSIDE STRAIN GAGES WERE ATTACHED AT THREE DIFFERENT SECTIONS.*
C    *  STRAINS WERE MEASURED IN MICRO STRAINS BY MEANS OF AUTOMATIC  *
C    *  MULTICHANNEL DIGITAL STRAIN INDICATOR.                        *
C    *  MATERIAL OF CONDUIT USED :- ALLUMINIUM ALLOY 6061-T6(EXTRUDED)*
C    *  MODULUS OF ELASTICITY OF CONDUIT MATERIAL =10*10**6 PSI       *
C    *  INSIDE DIAMETER OF CONDUIT= 31.0 INCH.                        *
C    *  THICKNESS OF CONDUIT WALL = 3/16 INCH.                        *
C    *  POISSON'S RATIO (NEU)=0.33                                    *
C    *  THE CONDUIT WAS TESTED FOR 1000. 1500. 2000 LBS CONCENTRATED  *
C    *  LOADS WITH ECCENTRICITIES 0.0, 7.50, 15.0 INCH. FOR EACH LOAD *
C    *  BASE PLATE SIZE USED = 8'' BY 11''                            *
C    *  SMALLER DIMENSION OF THE  BASE PLATE WAS KEPT PARALLEL TO THE *
C    *  LENGTH OF CONDUIT.                                            *
C    *  FOLLOWING FORMULAS ARE USED IN THRUST & MOMENT CALCULATIONS.  *
C    *  AVERAGE STRAIN=( OUTSIDE STRAIN +INSIDE STRAIN )/2.0          *
C    *  AXIAL THRUST (T)=E*AV.STRAIN*C/S AREA OF CONDUIT WALL/UNIT    *
C    *  LENGTH.                                                       *
C    *  STRAIN DUE TO BENDING=OUTSIDE STRAIN - AVERAGE STRAIN         *
C    *  STRESS DUE TO BENDING=BENDING STRAIN*E/(1-NEU*NEU)            *
C    *  BENDING MOMENT=BENDING STRESS*I/C  WHERE I IS THE  MOMENT OF  *
C    *  INERTIA OF THE  CONDUIT WALL & C=HALF THE  THICKNESS OF WALL. *
C    *  AVS=AVERAGE STRAIN OF OUTSIDE & INSIDE STRAIN GAGES.          *
C    *  SBEND=STRAIN DUE TO THE  BENDING.                             *
C    *  T,T1,T2=AXIAL THRUST IN LBS./UNIT LENGTH OF CONDUIT.          *
C    *  M,M1,M2=BENDING MOMENT IN IN.LB./UNIT LENGTH OF CONDUIT.      *
```

```
C    *
C    *   R.R1.R2=MCMENT TO THRUST RATIO.                                  *
C    *                                                                    *
C    *   IDEL & JDEL ARE STRAIN INDICATOR READINGS IN MICRC STRAIN.       *
C    *                                                                    *
C    *   DEL IS THE DIFFERENCE OF TWO CONSECUTIVE ZERC LOAD               *
C    *                                                                    *
C    *   STRAIN READINGS.                                                 *
C    *                                                                    *
C    *   RDEAL ARE CORRECTED STRAIN VALUES DUE TO LIVE LCAD ONLY.         *
C    *                                                                    *
C    *   TAV=TOTAL AVERAGE THRUST=AV.THRUST/UNIT WIDTH*TOTAL LENGTH       *
C    *                                                                    *
C    **********************************************************************
      DIMENSION IDEL(30.5.44).T(30.5.22).M(30.5.22).R(30.5.22).
     * JDEL(8.44).T1(30.5.22).M1(30.5.22).X1(5).RDEL(30.5.44)
     *.R1(30.5.22).DEL(30.5.44).IP(5).TC(8.22).MC(8.22).RC(8.22)
     *.T2(30.22).T3(3.22).M2(30.22).M3(3.22).R3(3.22).NDEL(8.44)
      DIMENSION TAV(4.10.5).VT(4.10.4)
      REAL M.M1.M2.M3.MC
C     JDEL(MY.I) ARE THE CONSTRUCTION STAGE STRAIN GAGE READINGS.
      DO 5 MY=1.8
    5 READ(5.15)(JDEL(MY.I).I=1.44)
      DO 10 MY=1.30
      DO 10 L=1.5
   10 READ(5.15) (IDEL(MY.L.I).I=1.44)
   15 FORMAT(20I4./.20I4./.4I4)
      DO 6 MY=1.8
      DO 7 I=1.22
      NDEL(MY.I)=JDEL(MY.I)-JDEL(1.I)
      NDEL(MY.I+22)=JDEL(MY.I+22)-JDEL(1.I+22)
      AVS=(NDEL(MY.I)+NDEL(MY.I+22))/2.0
      SBEND=NDEL(MY.I+22)-AVS
      DSIGMA=11.222085*AVS
      BSIGMA=11.222085*SBEND
      TC(MY.I)=3.0*DSIGMA/16.
      MC(MY.I)=BSIGMA/170.66667
      IF(TC(MY.I).EQ.0.0) GO TO 7
      RC(MY.I)=MC(MY.I)/TC(MY.I)
    7 CONTINUE
    6 CONTINUE
      DO 30 MY=1.30
      DO 35 L=1.5
      DO 40 I=1.44
      DEL(MY.L.I)=IDEL(MY.5.I)-IDEL(MY.1.I)
   40 CONTINUE
   35 CONTINUE
   30 CONTINUE
      X1(1)=0.00;X1(2)=0.25;X1(3)=0.50;X1(4)=0.75;X1(5)=1.00
      DO 55 MY=1.30
      DO 60 L=1.5
      DO 65 I=1.22
      RDEL(MY.L.I)=IDEL(MY.L.I)-(IDEL(MY.1.I)+DEL(MY.L.I)*X1(L))
      RDEL(MY.L.I+22)=IDEL(MY.L.I+22)-(IDEL(MY.1.I+22)+DEL(MY.L.I+22)
     **X1(L))
      AVS=(RDEL(MY.L.I)+RDEL(MY.L.I+22))/2.0
      SBEND=RDEL(MY.L.I+22)-AVS
      DSIGMA=11.222085*AVS
      BSIGMA=11.222085*SBEND
      T(MY.L.I)=3.0*DSIGMA/16.0
      M(MY.L.I)=BSIGMA/170.66667
      IF(T(MY.L.I).EQ.0.0) GO TO 65
      R(MY.L.I)=M(MY.L.I)/T(MY.L.I)
   65 CONTINUE
   60 CONTINUE
   55 CONTINUE
      DO 70 MY=1.30
      DO 75 I=1.22
      AVS=(IDEL(MY.1.I)+IDEL(MY.1.I+22))/2.0
      SBEND=IDEL(MY.1.I+22)-AVS
      DSIGMA=11.222085*AVS
      BSIGMA=11.222085*SBEND
      T1(MY.1.I)=3.0*DSIGMA/16.0
      M1(MY.1.I)=BSIGMA/170.66667
      IF(T1(MY.1.I).EQ.0.0) GO TO 75
      R1(MY.1.I)=M1(MY.1.I)/T1(MY.1.I)
   75 CONTINUE
   70 CONTINUE
      DO 76 MY=1.30
      DO 77 I=1.22
```

```
      AV1=(IDEL(MY,5,I)+IDEL(MY,5,I+22))/2.0
      SBEND1=IDEL(MY,5,I+22)-AV1
      OSIGM1=11.222085*AV1
      BSIGM1=11.222085*SBEND1
      T1(MY,5,I)=3.0*OSIGM1/16.0
      M1(MY,5,I)=BSIGM1/170.66667
      T2(MY,I)=(T1(MY,1,I)+T1(MY,5,I))/2.0
      M2(MY,I)=(M1(MY,1,I)+M1(MY,5,I))/2.0
      IF(T1(MY,5,I).EQ.0.0) GO TO 77
      R1(MY,5,I)=M1(MY,5,I)/T1(MY,5,I)
77    CONTINUE
76    CONTINUE
      DO 85 I=1,22
      T3(1,I)=(T2(1,I)+T2(2,I)+T2(3,I)+T2(4,I)+T2(5,I)+
     *T2(6,I)+T2(7,I)+T2(8,I)+T2(9,I))/9.0
      T3(2,I)=(T2(10,I)+T2(11,I)+T2(12,I)+T2(13,I)+
     *T2(14,I)+T2(15,I)+T2(16,I)+T2(17,I)+T2(18,I))/9.0
      T3(3,I)=(T2(19,I)+T2(20,I)+T2(21,I)+T2(22,I)+
     *T2(23,I)+T2(24,I)+T2(25,I)+T2(26,I)+T2(27,I))/9.0
      M3(1,I)=(M2(1,I)+M2(2,I)+M2(3,I)+M2(4,I)+M2(5,I)+
     *M2(6,I)+M2(7,I)+M2(8,I)+M2(9,I))/9.0
      M3(2,I)=(M2(10,I)+M2(11,I)+M2(12,I)+M2(13,I)+
     *M2(14,I)+M2(15,I)+M2(16,I)+M2(17,I)+M2(18,I))/9.0
      M3(3,I)=(M2(19,I)+M2(20,I)+M2(21,I)+M2(22,I)+
     *M2(23,I)+M2(24,I)+M2(25,I)+M2(26,I)+M2(27,I))/9.0
      IF(T3(1,I).EQ.0.0) GO TO 85
      R3(1,I)=M3(1,I)/T3(1,I)
85    CONTINUE
      DO 83 I=1,22
      IF(T3(2,I).EQ.0.0) GO TO 83
      R3(2,I)=M3(2,I)/T3(2,I)
83    CONTINUE
      DO 84 I=1,22
      IF(T3(3,I).EQ.0.0) GO TO 84
      R3(3,I)=M3(3,I)/T3(3,I)
84    CONTINUE
      WRITE(6,86)
86    FORMAT(//,130('.'),//,10X,'DISTRIBUTION OF THRUST & BENDING
     *MOMENT IN THE CONDUIT DURING CONSTRUCTION STAGE.',//,
     *130('.'))
      DO 87 MY=2,8
      IF(MY.EQ.2) H=7.75
      IF(MY.EQ.3) H=15.5
      IF(MY.EQ.4) H=23.25
      IF(MY.EQ.5) H=31.0
      IF(MY.EQ.6) H=36.C
      IF(MY.EQ.7) H=38.5
      IF(MY.EQ.8) H=41.C
      WRITE(6,91) H
91    FORMAT(//,10X,'..... DEPTH OF SOIL FROM BED = ',F5.2,1X,'IN......'
     *,//,8X,'LOCATION',15X,'AXIAL THRUST(LBS)',12X,'BENDING MOMENT
     *(IN.LB.)',10X,'M BY T RATIO',//)
      DO 88 I=1,22
89    WRITE(6,95)I,TC(MY,I),MC(MY,I),RC(MY,I)
88    CONTINUE
87    CONTINUE
      DO 90 MY=1,30
      IF(MY.LE.9) H=5.0
      IF(MY.GE.10.AND.MY.LE.18) H=7.5
      IF(MY.GE.19.AND.MY.LE.27) H=10.0
      IF(MY.GE.28) H=12.5
      DO 90 L=2,4
      IF(L.EQ.2) PB=100C.
      IF(L.EQ.3) PB=1500.
      IF(L.EQ.4) PB=200C.
      WRITE(6,100) MY,PB,H
100   FORMAT(//,130('*'),//,10X,'***** LOADING LOCATION= ',I2,
     *2X,'*****',//,10X,'***** LOAD IN LBS.= ',F5.1,2X,'*****',
     *//,10X,'***** SOIL COVER= ',F4.1,1X,'INCH.*****',//,
     *9X,'LOCATION',15X,'AXIAL THRUST(LBS)',12X,'BENDING MOMENT(IN.LB.)
     *',14X,'M/T RATIO ',//)
      DO 90 I=1,22
90    WRITE(6,95)I,T(MY,L,I),M(MY,L,I),R(MY,L,I)
95    FORMAT(/,10X,I2,21X,F8.3,21X,F8.3,25X,F9.5)
      WRITE(6,105)
105   FORMAT(//,130('*'),//,10X,'FOLLOWING ARE THE VALUES OF ',//,
     *10X,'T,M & R DUE TO SOIL WEIGHT ONLY ',//,130('*'))
      DO 110 MY=1,30
      IF(MY.LE.9) H=5.0
      IF(MY.GE.10.AND.MY.LE.18) H=7.5
```

```
      WRITE(6.220) H
      WRITE(6.231)
      DO 360 L=2.4
      N=(MY-1)*9+1
      WRITE(6.270) IP(L).T(N.L.10).T(N.L.21).T(N+1.L.2).
     *T(N+1.L.3).T(N+1.L.4).T(N+1.L.5).T(N+1.L.6).T(N+1.L.7).
     *T(N+1.L.8).T(N.L.11).
     *M(N.L.10).M(N.L.21).M(N+1.L.2).M(N+1.L.3).M(N+1.L.4).
     *M(N+1.L.5).M(N+1.L.6).M(N+1.L.7).M(N+1.L.8).M(N.L.11)
360   CONTINUE
350   CONTINUE
      WRITE(6.440)
440   FORMAT(//.130('.').//.5X.'..... ALONG LINE 1-1 IN LONGL. '.
     *'DIRECTION  .....'./)
      DO 450 MY=1.4
      IF(MY.EQ.1) H=5.0
      IF(MY.EQ.2) H=7.5
      IF(MY.EQ.3) H=10.0
      IF(MY.EQ.4) H=12.5
      WRITE(6.220) H
      WRITE(6.441)
441   FORMAT(/.5X.'LOAD'.7X.'E--E'.8X.'C--C'.5X.'A--A'.8X.'B--B'.
     *8X.'D--D'.8X.'T-AV.'.4X.'V-THRUST'.//)
      DO 460 L=2.4
      N=(MY-1)*9+1
      WRITE(6.470) IP(L).T(N.L.12).T(N.L.10).T(N.L.1).T(N.L.10).
     *T(N.L.12).
     *M(N.L.12).M(N.L.10).M(N.L.1).M(N.L.10).M(N.L.12)
470   FORMAT(/.5X.I4.5(5X.F7.2)./.9X.5(5X.F7.2)./)
460   CONTINUE
450   CONTINUE
      DO 700 MY=1.4
      N=(MY-1)*9+1
      DO 705 L=2.4
      TAV(MY.1.L)=11.*(T(N.L.12)+T(N.L.12))+8.*(T(N.L.10)+T(N.L.1)
     *+T(N+1.L.1))
      VT(MY.1.L)=0.0
      TAV(MY.2.L)=11.*(T(N+2.L.21)+T(N.L.22))+8.*(T(N.L.21)+T(N+1.L.21)
     *+T(N.L.21))
      VT(MY.2.L)=TAV(MY.2.L)*0.58778525
      TAV(MY.3.L)=11.*(T(N.L.13)+T(N.L.13))+8.*(T(N+2.L.2)+T(N.L.2)+
     *T(N+1.L.2))
      VT(MY.3.L)=TAV(MY.3.L)*0.95105652
      TAV(MY.4.L)=11.*(T(N.L.14)+T(N.L.14))+8.*(T(N+2.L.14)+T(N.L.3)+
     *T(N+1.L.3))
      TAV(MY.5.L)=11.*(T(N.L.15)+T(N.L.15))+8.*(T(N+2.L.15)+T(N.L.4)+
     *T(N+1.L.4))
      TAV(MY.6.L)=11.*(T(N.L.16)+T(N.L.16))+8.*(T(N+2.L.16)+T(N.L.5)+
     *T(N+1.L.5))
      VT(MY.6.L)=0.0
      TAV(MY.7.L)=11.*(T(N.L.17)+T(N.L.17))+8.*(T(N+2.L.17)+T(N.L.6)+
     *T(N+1.L.6))
      TAV(MY.8.L)=11.*(T(N.L.18)+T(N.L.18))+8.*(T(N+2.L.18)+T(N.L.7)+
     *T(N+1.L.7))
      TAV(MY.9.L)=11.*(T(N.L.19)+T(N.L.19))+8.*(T(N.L.8)+T(N+1.L.8)+
     *T(N+1.L.8))
      VT(MY.9.L)=TAV(MY.9.L)*0.95105652
      TAV(MY.10.L)=11.*(T(N.L.20)+T(N.L.20))+8.*(T(N.L.11)+
     *T(N+2.L.11)+T(N+1.L.9))
      VT(MY.10.L)=TAV(MY.10.L)*0.58778525
705   CONTINUE
700   CONTINUE
730   FORMAT(/.5X.I4.5(5X.F7.2).5X.F7.1./)
731   FORMAT(/.5X.I4.5(5X.F7.2).2(5X.F6.0)./)
      DO 710 MY=1.4
      IF(MY.EQ.1) H=5.0
      IF(MY.EQ.2) H=7.5
      IF(MY.EQ.3) H=10.0
      IF(MY.EQ.4) H=12.5
      WRITE(6.720) H
      N=(MY-1)*9+1
720   FORMAT(//.130(':').//.10X.'... SOIL COVER ON TOP OF CONDUIT = '
     *.F4.1.' INCHES ....'.//)
      WRITE(6.722)
722   FORMAT(//.130('.').//.10X.'ALONG LINE 1-1'.//)
      WRITE(6.441)
      DO 723 L=2.4
      WRITE(6.731) IP(L).T(N.L.12).T(N.L.10).T(N.L.1).T(N+1.L.1).
     *T(N.L.12).TAV(MY.1.L).VT(MY.1.L)
723   CONTINUE
```

```
      IF(MY.GE.19.AND.MY.LE.27) H=10.0
      IF(MY.GE.28) H=12.5
      WRITE(6,115) H,MY
115   FORMAT(//,10X,'***** SOIL COVER= ',F4.1,2X,'INCH. *****',//,
     *10X,'..... LOCATION MY = ',I2,2X,'.....',//,
     *3X,'LOCATION',5X,'T1(1)',7X,'M1(1)',9X,'R1(1)',12X,'T1(5)'
     *,7X,'M1(5)',8X,'R1(5)',10X,'T2(AV.)',7X,'M2(AV.)',//)
      DO 110 I=1,22
110   WRITE(6,96)I,T1(MY,1,I),M1(MY,1,I),R1(MY,1,I),T1(MY,5,I),
     *M1(MY,5,I),R1(MY,5,I),T2(MY,I),M2(MY,I)
96    FORMAT(/,5X,I2,2(7X,F8.3,5X,F8.3,5X,F9.5),2X,2(5X,F8.3))
      WRITE(6,120)
120   FORMAT(//,130('*'),//,
     *10X,'FOLLOWING ARE THE AVERAGE VALUES OF T,M & R DUE TO ',//,
     *10X,'SOIL WEIGHT ONLY, FOR DIFFERENT SOIL COVERS',//,130('*'),//)
      DO 125 MY=1,3
      IF(MY.EQ.1) H=5.0
      IF(MY.EQ.2) H=7.5
      IF(MY.EQ.3) H=10.0
      WRITE(6,130)H
130   FORMAT(//,10X,'***** SOIL COVER= ',F4.1,2X,'INCH. *****',//,
     *9X,'LOCATION',15X,'AXIAL THRUST(LBS)',12X,'BENDING MOMENT(IN.LB
     *.)',13X,'M BY T RATIO',//)
      DO 125 I=1,22
125   WRITE(6,95)I,T3(MY,I),M3(MY,I),R3(MY,I)
      IP(1)=0
      IP(2)=1000
      IP(3)=1500
      IP(4)=2000
      WRITE(6,200)
200   FORMAT(/,130('*'),//,43X,'FOLLOWING ARE THE THRUST AND MOMENT',
     *' VALUES WHEN ',///,43X,'LOAD IS ACTING AT AA(MIDSPAN) WITH ZERO',
     *'  ECC.',//,130('*'))
      WRITE(6,210)
210   FORMAT(//,52X,'..... ALONG SECTION AA .....',///)
      DO 225 MY=1,4
      IF(MY.EQ.1) H=5.0
      IF(MY.EQ.2) H=7.50
      IF(MY.EQ.3) H=10.0
      IF(MY.EQ.4) H=12.5
      WRITE(6,220) H
220   FORMAT(130('-'),//,50X,'..... SOIL COVER= ',F4.1,' INCH. .....',/)
      WRITE(6,231)
231   FORMAT(/,3X,'LOAD',6X,'AXIS1',6X,'AXIS2',6X,'AXIS3',6X,'AXIS4',
     *6X,'AXIS5',6X,'AXIS6',6X,'AXIS7',6X,'AXIS8',6X,'AXIS9',6X,
     *'AXIS10',/)
      DO 230 L=2,4
      N=(MY-1)*9+1
      WRITE(6,235) IP(L),T(N,L,1),T(N+2,L,11),T(N,L,2),T(N,L,3),
     *T(N,L,4),T(N,L,5),T(N,L,6),T(N,L,7),T(N,L,8),T(N,L,9),
     *M(N,L,1),M(N+2,L,11),M(N,L,2),M(N,L,3),M(N,L,4),M(N,L,5),
     *M(N,L,6),M(N,L,7),M(N,L,8),M(N,L,9)
235   FORMAT(/,3X,I4,10(4X,F7.2),/,7X,10(4X,F7.2),/)
230   CONTINUE
225   CONTINUE
      WRITE(6,240)
240   FORMAT(//,130('.'),//,50X,'..... ALONG SECTION EE & DD .....',///)
      DO 250 MY=1,4
      IF(MY.EQ.1) H=5.0
      IF(MY.EQ.2) H=7.5
      IF(MY.EQ.3) H=10.0
      IF(MY.EQ.4) H=12.5
      WRITE(6,220) H
      WRITE(6,231)
      DO 260 L=2,4
      N=(MY-1)*9+1
      WRITE(6,270) IP(L),T(N,L,12),T(N,L,22),T(N,L,13),T(N,L,14),
     *T(N,L,15),T(N,L,16),T(N,L,17),T(N,L,18),T(N,L,19),T(N,L,20),
     *M(N,L,12),M(N,L,22),M(N,L,13),M(N,L,14),M(N,L,15),M(N,L,16),
     *M(N,L,17),M(N,L,18),M(N,L,19),M(N,L,20)
270   FORMAT(/,3X,I4,10(4X,F7.2),/,7X,10(4X,F7.2),/)
260   CONTINUE
250   CONTINUE
      WRITE(6,340)
340   FORMAT(//,130('.'),//,50X,'..... ALONG SECTION BB & CC .....',///)
      DO 350 MY=1,4
      IF(MY.EQ.1) H=5.0
      IF(MY.EQ.2) H=7.5
      IF(MY.EQ.3) H=10.0
      IF(MY.EQ.4) H=12.5
```

```
      WRITE(6,721)
721   FORMAT(//,130('.'),//,10X,'ALONG LINE 2-2',//)
      WRITE(6,441)
      DO 725 L=2,4
      WRITE(6,731)  IP(L),T(N+2,L,21),T(N,L,21),T(N+1,L,21),T(N,L,21),
     *T(N,L,22),TAV(MY,2,L),VT(MY,2,L)
725   CONTINUE
      WRITE(6,726)
726   FORMAT(//,130('.'),//10X,'ALONG LINE 3-3',//)
      WRITE(6,441)
      DO 735 L=2,4
      WRITE(6,731)  IP(L),T(N,L,13),T(N+2,L,2),T(N,L,2),T(N+1,L,2),
     *T(N,L,13),TAV(MY,3,L),VT(MY,3,L)
735   CONTINUE
      WRITE(6,736)
736   FORMAT(//,130('.'),//,10X,'ALONG LINE 4-4',//)
      WRITE(6,441)
      DO 740 L=2,4
      WRITE(6,730)  IP(L),T(N,L,14),T(N+2,L,14),T(N,L,3),T(N+1,L,3),
     *T(N,L,14),TAV(MY,4,L)
740   CONTINUE
      WRITE(5,741)
741   FORMAT(//,130('.'),//,10X,'ALONG LINE 5-5',//)
      WRITE(6,441)
      DO 745 L=2,4
      WRITE(6,730)  IP(L),T(N,L,15),T(N+2,L,15),T(N,L,4),T(N+1,L,4),
     *T(N,L,15),TAV(MY,5,L)
745   CONTINUE
      WRITE(6,746)
746   FORMAT(//,130('.'),//,10X,'ALONG LINE 6-6',//)
      WRITE(6,441)
      DO 750 L=2,4
      WRITE(6,731)  IP(L),T(N,L,16),T(N+2,L,18),T(N,L,5),T(N+1,L,5),
     *T(N,L,16),TAV(MY,6,L),VT(MY,6,L)
750   CONTINUE
      WRITE(6,751)
751   FORMAT(//,130('.'),//,10X,'ALONG LINE 7-7',//)
      WRITE(6,441)
      DO 755 L=2,4
      WRITE(6,730)  IP(L),T(N,L,17),T(N+2,L,17),T(N,L,6),T(N+1,L,6),
     *T(N,L,17),TAV(MY,7,L)
755   CONTINUE
      WRITE(6,756)
756   FORMAT(//,130('.'),//,10X,'ALONG LINE 8-8',//)
      WRITE(5,441)
      DO 760 L=2,4
      WRITE(6,730)IP(L),T(N,L,18),T(N+2,L,18),T(N,L,7),T(N+1,L,7),
     *T(N,L,18),TAV(MY,8,L)
760   CONTINUE
      WRITE(6,761)
761   FORMAT(//,130('.'),//,10X,'ALONG LINE 9-9',//)
      WRITE(6,441)
      DO 765 L=2,4
      WRITE(6,731)  IP(L),T(N,L,19),T(N+1,L,8),T(N,L,8),T(N+1,L,8),
     *T(N,L,19),TAV(MY,9,L),VT(MY,9,L)
765   CONTINUE
      WRITE(6,766)
766   FORMAT(//,130('.'),//,10X,'ALONG LINE 10-10',//)
      WRITE(6,441)
      DO 770 L=2,4
      WRITE(6,731)  IP(L),T(N,L,20),T(N,L,11),T(N+2,L,11),T(N,L,11),
     *T(N+1,L,11),TAV(MY,10,L),VT(MY,10,L)
770   CONTINUE
710   CONTINUE
      STOP
      END

$ENTRY
```

FOLLOWING ARE THE THRUST AND MOMENT VALUES WHEN

LOAD IS ACTING AT AA(MIDSPAN) WITH ZERO ECC.

..... ALONG SECTION AA .....

..... SOIL COVER= 5.0 INCH. .....

LOAD	AXIS1	AXIS2	AXIS3	AXIS4	AXIS5	AXIS6	AXIS7	AXIS8	AXIS9	AXIS10
1000	-117.83 21.60	-79.01 -13.34	-11.31 2.57	-0.79 -0.06	-3.48 -0.01	-0.79 0.40	-1.09 0.08	1.84 -0.29	-11.87 2.27	-88.22 -12.14
1500	-206.21 38.14	-122.57 -22.80	-16.31 4.03	1.58 -0.28	-1.38 -0.12	0.53 1.13	-2.10 -0.03	3.68 -0.14	-17.89 3.39	-122.87 -20.33
2000	-327.19 60.49	-168.07 -32.95	-21.30 5.09	0.79 -0.34	-3.95 0.04	-0.24 1.93	-4.21 -0.08	4.47 3.09	-26.30 3.98	-176.48 -30.07

..... SOIL COVER= 7.5 INCH. .....

LOAD	AXIS1	AXIS2	AXIS3	AXIS4	AXIS5	AXIS6	AXIS7	AXIS8	AXIS9	AXIS10
1000	-71.40 18.17	-66.28 -9.95	-14.20 1.23	-4.73 -0.13	-4.47 -0.08	-2.53 0.44	-1.32 3.18	-3.68 0.12	-10.26 1.64	-68.12 -9.38
1500	-119.20 28.82	-102.09 -16.90	-21.04 2.04	-2.10 -0.10	-3.66 -0.02	-1.05 3.69	2.53 1.15	-2.10 3.07	-9.99 2.88	-99.42 -15.73
2000	-177.54 46.53	-145.19 -25.20	-28.93 2.75	-3.68 0.20	-6.05 0.02	-2.63 3.87	-1.48 3.21	-1.38 0.12	-22.38 3.59	-149.66 -22.87

..... SOIL COVER= 10.0 INCH. .....

LOAD	AXIS1	AXIS2	AXIS3	AXIS4	AXIS5	AXIS6	AXIS7	AXIS8	AXIS9	AXIS10
1000	-36.03 8.98	-53.13 -8.58	-12.10 3.23	-1.32 -0.01	2.37 -3.07	3.00 3.00	2.63 3.07	1.05 -0.05	-13.41 3.62	-52.34 -5.53
1500	-62.60 19.63	-84.17 -11.80	-16.63 0.56	-2.63 0.06	-0.53 0.02	-1.05 3.16	2.10 0.16	1.05 0.00	-19.48 1.20	-85.74 -10.06
2000	-84.95 24.58	-119.41 -17.49	-18.41 0.99	-1.84 3.41	-1.32 0.04	0.00 3.33	1.58 0.20	-1.35 0.12	-28.36 1.75	-120.20 -15.18

..... SOIL COVER= 12.5 INCH. .....

LOAD	AXIS1	AXIS2	AXIS3	AXIS4	AXIS5	AXIS6	AXIS7	AXIS8	AXIS9	AXIS10
1000	-29.72 20.06	-45.80 -8.47	-8.15 0.46	1.32 0.29	0.60 0.16	2.37 0.37	-6.79 0.16	0.26 0.12	-6.31 0.71	-52.60 -8.12
1500	-60.49 21.32	-73.12 -8.99	-15.46 0.84	-3.60 0.25	-1.09 0.10	-0.53 3.38	-0.53 3.15	-1.88 0.14	-13.68 1.12	-75.75 -9.60
2000	-88.43 22.94	-102.86 -13.37	-28.67 3.08	-3.42 0.11	1.09 0.07	1.66 0.49	1.84 3.21	-0.26 0.21	-18.64 1.40	-97.84 -11.84

DEFLECTION – CONDUIT 2

```
$JOB     /ATFIV   XXXXXXXXXX    EKHANDE
         DIMENSION H(4),A(4),P(3),X(4,3),Y3(4),Y2(4),Z1(4,3,35),Z3(4,3,35),
        *Z2(4,3,35),DX1(4,3,35),DX2(4,3,35),IDEF(4,3),DX3(4,3,35)
         DO 5 I=1,4
    5    READ(5,6)(IDEF(I,J),J=1,3)
    6    FORMAT(3I4)
         READ(5,15)(H(I),I=1,4)
   15    FORMAT(4F5.2)
         READ(5,25) (P(I),I=1,3)
   25    FORMAT(3F5.0)
         READ(5,35) (A(I),I=1,4)
C        A(I) VALUES ARE CALCULATED BY ASSUMING 2:1 LOAD DISPERSION IN
C        THE LONGITUDINAL DIRECTION OF THE CONDUIT.:-OHBD CODE.
   35    FORMAT(4F5.2)
         DO 40 I=1,4
         Y2(I)=1-(15.5/(15.5+H(I)))**2
         Y3(I)=1-(15.5/(15.5+H(I)))**3
         DO 45 J=1,3
         X(I,J)=309081*(P(J)/A(I))
         DO 50 K=10,30
         Z1(I,J,K)=5494.+22715.64*K
         Z2(I,J,K)=5494+22715.64*K*Y2(I)
         Z3(I,J,K)=5494+22715.64*K*Y3(I)
C        DX1(I,J,K)=DEFLECTIONS CALCULATED BY ORIGINAL IOWA FORMULA.
C        DX2(I,J,K)=DEFLECTIONS CALCULATED BY CONSIDERING 2 ND DEGREE
C                   REDUCTION IN THE MODULUS OF SOIL REACTION.
C        DX3(I,J,K)=DEFLECTIONS CALCULATED BY CONSIDERING 3 RD DEGREE
C                   REDUCTION IN THE MODULUS OF SOIL REACTION.
         DX1(I,J,K)=X(I,J)/Z1(I,J,K)
         DX2(I,J,K)=X(I,J)/Z2(I,J,K)
         DX3(I,J,K)=X(I,J)/Z3(I,J,K)
   50    CONTINUE
   45    CONTINUE
   40    CONTINUE
         WRITE(6,36)
   36    FORMAT(/,130('='),///,17X,'FOLLOWING ARE THE MEASURED DEFLECTION'
        *,'S AT THE CROWN OF THE CONDUIT#2',///)
         WRITE(6,37)
   37    FORMAT(20X,'BACKFILL IN.',5X,'P=1000 LBS.',5X,'P=1500 LaS.',
        *5X,'P=2000 LBS.',/)
         DO 41 I=1,4
   41    WRITE(6,42)H(I),(IDEF(I,J),J=1,3)
   42    FORMAT(22X,F4.1,3(12X,I4),//)
         DO 55 I=1,4
         WRITE(6,60) H(I)
   60    FORMAT(//,130('.'),///,38X,'..... SOIL COVER = ',F4.1,' INCH.....'
        *,///,27X,'..... LOAD P=1000 LBS.....',5X,'..... LOAD P=1500 LBS'
        *,'.....',6X,'..... LOAD P=2000 LBS.....',//)
         WRITE(6,65)
   65    FORMAT(5X,'NO.',2X,'MS REACTION',3(7X,'DELTA-1',2X,'DELTA-2',2X
        *,'DELTA-3'),//)
         DO 55 K=10,30
         KK=100*K
         KL=K-9
   55    WRITE(6,70)KL,KK,(DX1(I,J,K),DX2(I,J,K),DX3(I,J,K),J=1,3)
   70    FORMAT(5X,I2,6X,I5,10X,F5.0,5X,F5.0,4X,F5.0,9X,F5.0,4X,F5.0,
        *4X,F5.0,8X,F5.0,4X,F5.0,5X,F5.0,/)
         WRITE(6,7)
    7    FORMAT(/,130('='),/)
         STOP
         END

$ENTRY
```

FOLLOWING ARE THE MEASURED DEFLECTIONS AT THE CROWN OF THE CONDUIT#2

BACKFILL IN.	P=1000 LBS.	P=1500 LBS.	P=2000 LBS.
5.0	90	152	248
7.5	65	107	170
10.0	40	60	98
12.5	28	42	55

..... SOIL COVER = 5.0 INCH.....

NO.	KS REACTION	LOAD P=1000 LBS DELTA-1	LOAD P=1000 LBS DELTA-2	LOAD P=1000 LBS DELTA-3	LOAD P=1500 LBS DELTA-1	LOAD P=1500 LBS DELTA-2	LOAD P=1500 LBS DELTA-3	LOAD P=2000 LBS DELTA-1	LOAD P=2000 LBS DELTA-2	LOAD P=2000 LBS DELTA-3
1	1000	162.	367.	288.	243.	550.	420.	324.	733.	561.
2	1100	148.	338.	256.	221.	508.	384.	298.	670.	512.
3	1200	136.	308.	232.	203.	462.	353.	271.	617.	470.
4	1300	128.	286.	218.	188.	428.	327.	251.	571.	436.
5	1400	117.	266.	203.	175.	399.	304.	233.	532.	405.
6	1500	109.	249.	189.	163.	373.	284.	218.	498.	379.
7	1600	102.	234.	178.	153.	351.	267.	204.	468.	356.
8	1700	96.	221.	168.	144.	331.	252.	192.	441.	336.
9	1800	91.	209.	159.	136.	313.	238.	182.	417.	317.
10	1900	86.	198.	150.	129.	297.	226.	172.	396.	301.
11	2000	82.	188.	143.	123.	283.	215.	164.	377.	286.
12	2100	78.	180.	136.	117.	269.	205.	156.	359.	273.
13	2200	75.	172.	130.	112.	258.	196.	149.	343.	261.
14	2300	71.	164.	125.	107.	247.	187.	143.	329.	250.
15	2400	68.	158.	120.	103.	237.	179.	137.	316.	239.
16	2500	66.	152.	115.	99.	227.	172.	131.	303.	230.
17	2600	63.	146.	111.	95.	219.	166.	126.	292.	221.
18	2700	61.	141.	107.	91.	211.	160.	122.	281.	213.
19	2800	59.	136.	103.	88.	203.	154.	118.	271.	206.
20	2900	57.	131.	99.	85.	197.	149.	113.	262.	199.
21	3000	55.	127.	96.	82.	190.	144.	110.	254.	192.

..... SOIL COVER = 7.5 INCH.....

NO.	KS REACTION	LOAD P=1000 LBS DELTA-1	LOAD P=1000 LBS DELTA-2	LOAD P=1000 LBS DELTA-3	LOAD P=1500 LBS DELTA-1	LOAD P=1500 LBS DELTA-2	LOAD P=1500 LBS DELTA-3	LOAD P=2000 LBS DELTA-1	LOAD P=2000 LBS DELTA-2	LOAD P=2000 LBS DELTA-3
1	1000	124.	223.	177.	186.	335.	266.	248.	446.	354.
2	1100	113.	204.	161.	170.	305.	242.	226.	407.	323.
3	1200	104.	187.	148.	156.	281.	223.	208.	374.	297.
4	1300	96.	173.	137.	144.	260.	206.	192.	347.	275.
5	1400	89.	161.	128.	134.	242.	192.	179.	323.	256.
6	1500	83.	151.	119.	125.	226.	179.	167.	302.	239.
7	1600	78.	142.	112.	117.	213.	168.	157.	283.	224.
8	1700	74.	134.	106.	111.	200.	158.	148.	267.	211.
9	1800	70.	126.	100.	105.	189.	150.	139.	253.	200.
10	1900	66.	120.	95.	99.	180.	142.	134.	240.	189.
11	2000	63.	114.	90.	94.	171.	135.	126.	228.	180.
12	2100	60.	109.	86.	90.	163.	129.	120.	217.	172.
13	2200	57.	104.	82.	86.	156.	123.	114.	208.	164.
14	2300	55.	99.	78.	82.	149.	118.	109.	199.	157.
15	2400	52.	95.	75.	79.	143.	113.	105.	191.	151.
16	2500	50.	92.	73.	76.	137.	108.	101.	183.	145.
17	2600	48.	88.	70.	73.	132.	104.	97.	176.	139.
18	2700	47.	85.	67.	70.	127.	101.	93.	170.	134.
19	2800	45.	82.	65.	68.	123.	97.	90.	164.	129.
20	2900	43.	79.	63.	65.	119.	94.	87.	158.	125.
21	3000	42.	77.	60.	63.	115.	91.	84.	153.	121.

..... SOIL COVER = 10.0 INCH.....

NO.	MS REACTION	..... LOAD P=1000 LBS..... DELTA-1	DELTA-2	DELTA-3	..... LOAD P=1500 LBS..... DELTA-1	DELTA-2	DELTA-3	..... LOAD P=2000 LBS..... DELTA-1	DELTA-2	DELTA-3
1	1000	161.	187.	129.	181.	336.	193.	201.	315.	256.
2	1100	92.	144.	116.	138.	215.	176.	163.	267.	238.
3	1200	84.	132.	108.	126.	198.	162.	166.	260.	216.
4	1300	76.	122.	100.	117.	183.	150.	156.	244.	200.
5	1400	72.	114.	93.	109.	176.	139.	145.	227.	186.
6	1500	68.	106.	87.	101.	169.	128.	135.	213.	174.
7	1600	63.	100.	81.	95.	158.	122.	127.	200.	163.
8	1700	58.	94.	77.	90.	141.	115.	120.	188.	154.
9	1800	57.	90.	73.	86.	133.	109.	113.	178.	146.
10	1900	54.	86.	69.	80.	127.	103.	107.	169.	138.
11	2000	51.	80.	65.	78.	120.	98.	104.	160.	131.
12	2100	49.	76.	62.	73.	115.	94.	97.	153.	125.
13	2200	46.	73.	60.	70.	110.	89.	93.	146.	119.
14	2300	44.	70.	57.	67.	105.	86.	89.	140.	114.
15	2400	43.	67.	55.	64.	101.	82.	88.	134.	109.
16	2500	41.	64.	53.	61.	97.	79.	82.	129.	105.
17	2600	39.	62.	51.	59.	93.	76.	79.	124.	101.
18	2700	38.	60.	49.	57.	90.	73.	76.	119.	97.
19	2800	36.	58.	47.	55.	88.	70.	73.	115.	94.
20	2900	35.	56.	45.	53.	83.	68.	71.	111.	91.
21	3000	34.	54.	44.	51.	81.	66.	68.	106.	88.

..... SOIL COVE[illegible]2.5 INCH.....

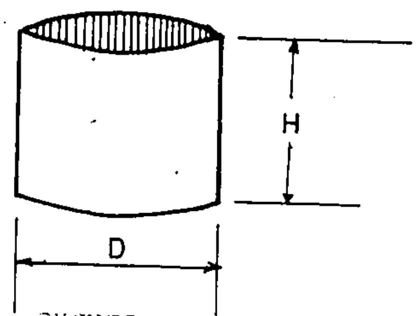
NO.	MS REACTION	..... LOAD P=1000 LBS..... DELTA-1	DELTA-2	DELTA-3	..... LOAD P=1500 LBS..... DELTA-1	DELTA-2	DELTA-3	..... LOAD P=2000 LBS..... DELTA-1	DELTA-2	DELTA-3
1	1000	88.	131.	161.	127.	181.	152.	169.	241.	203.
2	1100	77.	110.	92.	116.	168.	139.	154.	220.	188.
3	1200	71.	101.	85.	106.	152.	127.	142.	202.	170.
4	1300	65.	94.	79.	98.	140.	116.	131.	187.	157.
5	1400	61.	87.	73.	91.	131.	110.	122.	174.	146.
6	1500	57.	81.	68.	86.	123.	102.	114.	162.	137.
7	1600	53.	76.	64.	80.	115.	96.	107.	153.	128.
8	1700	50.	72.	60.	75.	108.	91.	101.	144.	121.
9	1800	48.	68.	57.	71.	102.	86.	95.	136.	114.
10	1900	45.	65.	54.	68.	97.	81.	90.	129.	108.
11	2000	43.	61.	51.	64.	92.	77.	86.	123.	103.
12	2100	41.	59.	49.	61.	88.	74.	82.	117.	98.
13	2200	39.	56.	47.	58.	84.	70.	78.	112.	94.
14	2300	37.	54.	45.	56.	80.	67.	75.	107.	90.
15	2400	36.	51.	43.	54.	77.	64.	72.	103.	86.
16	2500	34.	49.	41.	52.	74.	62.	69.	98.	83.
17	2600	33.	47.	40.	50.	71.	60.	66.	95.	79.
18	2700	32.	46.	38.	48.	69.	57.	64.	91.	76.
19	2800.	31.	44.	37.	46.	66.	55.	61.	88.	74.
20	2900	30.	43.	36.	44.	64.	53.	59.	85.	71.
21	3000	29.	41.	34.	43.	62.	52.	57.	82.	69.

ECCENTRICITY "e" Inch.	CONDUIT # 1									CONDUIT # 2											
	H = 5"			H = 7.5"			H = 10"			H = 5"			H = 7.5"			H = 10"			H = 12.5"		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
0	1	4	7	10	13	16	19	22	25	1	2	3	10	11	12	19	20	21	28	29	30
7.5	2	5	8	11	14	17	20	23	26	4	5	6	13	14	15	22	23	24	-	-	-
15	3	6	9	12	15	18	21	24	27	7	8	9	16	17	18	25	26	27	-	-	-

K - IDENTIFICATION NUMBER

A P P E N D I X :- C

DENSITY OF LOOSE AND COMPACTED SOIL.



$H = 11\frac{1}{4}$ IN.
$D = 14$ IN.

BUCKET

W_b=Weight of bucket =25.4 lb.
W_l=Loose soil weight; lb.
W_c=Compacted soil weight; lb.
γ_L=Loose soil density; pcf.
γ_C=Compacted soil density; pcf.
V_b=Volume of bucket $=\pi/4 \cdot D^2 \cdot H = 1.0$ ft.^{3}

NO.	W_L+W_b lb	W_c+W_b lb	W_L lb	W_c lb	γ_L pcf	γ_C pcf
1	141.4	145.3	116.0	119.9	116.0	119.9
2	140.2	143.9	114.8	118.5	114.8	118.5
3	142.2	145.3	116.8	119.9	116.8	119.9
4	140.7	143.7	115.3	118.3	115.3	118.3
5	142.9	146.0	117.5	120.6	117.5	120.5
Average loose & compacted unit Wt.					116.1	119.4

APPENDIX-C :- LOOSE AND COMPACTED SOIL DENSITY.

VITA AUCTORIS

VITA AUCTORIS

SHANTARAM G. EKHANDE

1953 Born on 1st June in Pimpalner, Dhule (India).

1970 Matriculated from New English School, Pimpalner, Dhule (India).

1972 Completed two years of science course at Pratap College Amalner, Jalgaon (India).

1972 Winner of Sir Dorabji Tata Trust Scholarship India.

1973 Winner of Civil Engineering Merit Scholarship by Walchand Hirachand Charitable Trust, Bombay, India.

1976 In June, graduated with Bachelor of Engineering in Civil Engineering from the University of Poona, Poona (India).

1976 In July, joined M/S Tata Consulting Engineers, Bombay, India as a Trainee Engineer.

1976 In September, joined M/S Engineers India Limited, New Delhi, India as an Assistant Engineer.

1977 Passed the Indian Engineering Services (IES) examination, India.

1979 Awarded J.N. Tata Endowment Scholarship, Bombay, India for higher studies.

1980 In January, enrolled as a graduate student to pursue Master's degree in Civil Engineering at the University of Windsor, Windsor, Ontario, Canada.

1980 Joined as Research and Teaching Assistant in the Department of Civil Engineering, University of Windsor, Windsor, Ontario, Canada.